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# A TEXT-BOOK

ON

# ROOFS AND BRIDGES.

## PART III.

## BRIDGE DESIGN.

BY

MANSFIELD MERRIMAN,

PROFESSOR OF CIVIL ENGINEERING IN LEHIGH UNIVERSITY,

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# PREFACE.

In view of the completion of this series by the publication of Part IV, it has been thought best to reduce the size of the volume on Bridge Design so that the four parts may be uniform in price and nearly equal in size. Accordingly, this edition of Part III includes only Chapters I-XI of the former editions, the omitted matter being that which can best be spared in giving instruction in the class room. It is believed by the authors that it is more profitable for students to inspect bridges than to read descriptions of them, particularly in gaining a knowledge of details, and that the processes and principles of design set forth in this volume, if supplemented by such field work, will lead to a thorough fundamental knowledge of modern economic practice.

February 1, 1898.

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# BRIDGE DESIGN.

## CHAPTER I.

### HISTORY AND LITERATURE.

#### ART. 1. BRIDGE DESIGN PRIOR TO 1800.

Bridge design prior to the year 1800, and indeed for many years after, was almost wholly empirical. Arch bridges of stone had been successfully built since the time of the Romans, and structures of timber were used for roofs and often for bridges, but the true idea of a bridge truss and of the functions of its members was not fully developed until near the middle of the present century. The oldest wooden bridge on record is the famous "Pons Sublicius" built across the Tiber at Rome in 621 B.C., but this, like CÆSAR'S bridge across the Rhine in 55 B.C., is believed to have been merely a pile or trestle structure. The great bridge built by TRAJAN over the Danube, 105 A.D., consisted of wooden arches, some of which were 180 feet in span.

A bridge truss is a structure whose members are so arranged that they are subject only to longitudinal stresses of tension and compression, and which exerts only vertical pressures on the supporting piers or abutments. The members should be arranged in triangular figures so that no distortion of the structure can occur without bringing the proper stresses into action, and the applied loads should preferably be concentrated at the joints (Part I, Art. 23). A simple truss is one supported at the two ends, and these alone will be considered in this volume, as they constitute the great majority of bridge structures and are sufficient to fully illustrate almost all the principles governing details.

The king-post truss shown at *a* in Fig. 1 may be supposed to be the origin of all modern bridge trusses. Prior to 1800,

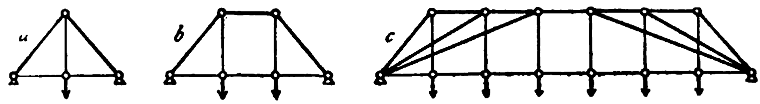



FIG. 1.

however, the principal line of development was that indicated by the diagrams *b* and *c*. On this plan many wooden bridges were erected during the seventeenth and eighteenth centuries. There were two chords, usually with a high camber, connected by vertical timbers acting as ties to support the floor which was placed along the lower chord. From the top of each vertical an inclined brace was carried to the nearest abutment and the tops of the corresponding pairs connected by a straining beam. True truss action as we now understand it scarcely existed, the main idea being to carry each load to the abutment by the shortest route. This was a simple plan, but it proved uneconomical on account of the long braces whose stresses increase both with their length and the angle of inclination to the vertical. On this plan was built in 1760 by GRUBENMANN a timber bridge near Baden, which had the great span of 366 feet, and which exhibited much skill in carpentry.

The secret of economical and efficient truss arrangement lies in the panel system, which may be regarded as having been developed from the king-post truss in the manner shown in Fig. 2, where *d* is derived from *a* by the addition of a panel on each side, and *e* from *d* in like manner. This system was first used by PALLADIO, an Italian architect, about 1570, but it produced little or no influence on methods of construction, until it was rediscovered and used in the United States near the close of the eighteenth century by THEODORE BURR. The Burr truss may indeed be called the parent of nearly all the forms of bridge trusses now used in this country. Although so defective

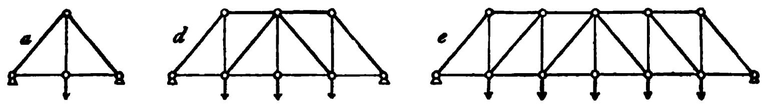



FIG. 2.

from the lack of counterbraces that it generally required the assistance of an arch to stiffen it under rolling loads, yet as it contained the sound principle of economy in a constant angle for the inclined members its panel system was transmitted to the Long truss, the Howe truss, and later to many other forms (Part I, Art. 25).

Concerning early timber bridges, as also for other valuable historical and descriptive matter, the student should consult the article Bridge in the Encyclopædia Britannica, the article Bridges in JOHNSON'S Universal Cyclopædia, 1893, and COOPER'S American Railroad Bridges, 1890.

## ART. 2. PROGRESS FROM 1800 TO 1850.

Near the beginning of the present century many wooden bridges were erected in the eastern and middle states by THEODORE BURR and by TIMOTHY PALMER, both of whom used the panel system. PALMER'S bridges generally combined the action of the truss and the arch in one structure, the lower chord being highly cambered, while BURR used the arch merely as auxiliary to the truss. The oldest bridge now standing in the United States is that built by BURR at Waterford, N. Y., in 1804, which is of hewn yellow pine, having four spans of 154, 161, 176, and 180 feet in the clear. The next oldest is that built by PALMER at Easton, Pa., in 1805, which has three spans of 195 feet between centers of piers. Illustrations of both of these bridges are given in COOPER'S American Railroad

Bridges. WERNWAG'S great bridge of 340 feet span, built at Philadelphia in 1812, also deserves notice as a splendid example of early engineering work; the double diagonal bracing in its panels showing that probably its builder had considered the distorting action of rolling loads.

The lattice truss introduced by TOWN about 1820 consisted of planks pinned together, and was important only on account of ease of construction. In 1830, however, a radical advance was made in the true principles of truss arrangement through the introduction of panel counterbraces by S. H. LONG. In a pamphlet published by him in 1836 the function of counterbraces in preventing the distortion of the panels under rolling loads, and also their use in stiffening the truss when keyed up so as to be under initial stress, is clearly recognized. Wooden Long trusses were erected on the Baltimore and Ohio Railroad as well as many for highway service.

In 1840 WILLIAM HOWE patented a combination truss having wooden chords and web diagonals and wrought-iron vertical ties, which has since been extensively used. Each panel had counter as well as main struts, both butting against cast-iron angle blocks. Many important bridges were built on this plan prior to 1850, the most notable being that over the Susquehanna at Havre de Grace, Md., which had thirteen spans of 250 feet each, and a draw span of shorter length. The Howe truss is still in common use in localities where timber is cheap, and for short spans and light traffic it often makes an efficient and economical bridge.

In 1844 the Pratt truss was introduced. In this a radical departure was made in the arrangement of the web members, the timber verticals being made to take compression, and the iron diagonals to take tension. This was a move in the direction of economy, since the length of the struts was decreased and thus the necessary cross-section somewhat diminished. Although at first built as a combination bridge, it never attained great popularity in this form, but soon after 1850 it began to be constructed entirely in iron, and in this form it has probably been more extensively used than any other form of truss.

But few iron structures were built in the United States prior to 1850, the first one being a span of 77 feet erected in 1840 over the Erie Canal, which was formed of cast-iron girders strengthened by wrought-iron rods. About the same time WHIPPLE built a truss with a curved upper chord of cast iron and a straight lower chord of wrought iron, forming the bowstring truss. A Howe truss in iron was introduced in 1844, and the Rider iron truss with a multiple web system was first built about 1847, but neither came into general use, and some that were built failed.

The first rational discussion of the determination of stresses and proportioning of cross-sections of truss members was published in 1847 at Utica, N. Y., by SQUIRE WHIPPLE under the title A Work on Bridge Building, in which are given methods of computing stresses due to dead and live loads, investigations as to the angle of economy for web bracing, with plans and details of the bowstring truss and of the double-system Pratt truss, since known as the Whipple truss. WHIPPLE was far in advance of his time in rational views of economic truss design, but the circulation of his book was small, so that its influence was not fully exerted until several years after publication. He also built over twenty bridges on his plans which gave good service for many years. SQUIRE WHIPPLE is justly regarded as the father of American rational bridge design. Drawings of bridges built between 1840 and 1850 may be seen in DUGGAN'S Stone, Iron, and Wood Bridges of United States Railroads, 1850; and also in HAUPT'S General Theory of Bridge Construction, 1851.

## ART. 3. TRUSS DEVELOPMENT SINCE 1850.

The modern bridge truss is the result of an evolution or development in the sense that it exhibits those features which experience has found to be most economical and stable. Forms costly or unsafe have disappeared, while those cheap and strong have remained in use. Thus, the panel system has survived, while the method of transferring loads directly to the abutments by long braces, as seen in Fig. 1, has gone out of use. The Bollman truss, introduced soon after 1850, was an instance of the application of that erroneous principle, but it could not be built for spans greater than 160 feet, and even for shorter spans it was unable to compete in economy and stability with trusses of the panel system. The Fink truss (Part I, Art. 53) is another example of the use of that principle, and it too has disappeared.

The Whipple truss (Fig. 3) is an instructive instance of a form which was extensively built from 1850 to 1885, even for

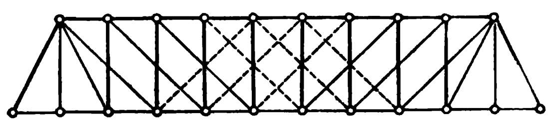


FIG. 3.

the longest spans, but which now is going out of use. This has all the advantages of the Pratt type as regards the use of vertical compression members in the web, and also by the double system of webbing the panel points are brought nearer together, thus decreasing the length of the stringers, which for long spans is a matter of importance. Stringers longer than 25 feet make an expensive floor, and this limits the economic depth of the Pratt truss to about 30 feet and the span to about 300 feet, since it is not advisable to make the depth less than one tenth of the span. With the Whipple truss, however, using the same angle for the bracing, the depth of the truss can be doubled and the span thus be economically increased. Many long bridges have been erected on this plan, among which may be mentioned the 515-feet span of the Ohio River bridge at Cincinnati, completed in 1877, and which at that date was the longest truss span ever erected. The Whipple truss began to go out of use merely because it was found to be more economical to support the floor beams by short sub-verticals attached to a single system of bracing than by the use of a double system, and because the single system is always more reliable and determinate in respect to stresses. The Post truss (Part I, Art. 55) is another example of a form once popular and now no longer in use.

The Warren or triangular truss was built with both single and double systems of webbing, but with a single system it afforded opportunity for the support of intermediate floor beams in a panel by the use of independent vertical members. In 1869 the channel span of 390 feet over the Ohio at Louisville was built on this plan, and in 1885 the 525-feet span at Henderson. This plan has been found advantageous because simplicity of truss action is secured, the only apparent disadvantage being the use of long inclined compression members in the webbing; in accordance with the law of evolution the former of these tends to be perpetuated and the latter to disappear.

At the present time the Pratt truss is most generally used for short spans. The Baltimore truss (Fig. 4) is used for both

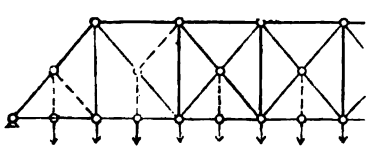


FIG. 4.

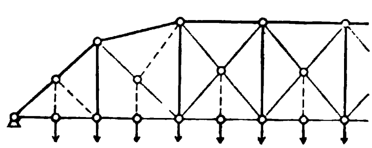


FIG. 5.

short and long spans; it possesses all the advantages of the Pratt, and in addition that of supporting intermediate floor beams by the use of sub-verticals. The modified bowstring truss, shown in Fig. 5, uses the same idea, and here are gained the important advantages that the stresses in the chords are rendered closely uniform, as also those in the webbing. These elements combined have rendered this form applicable to the longest simple trusses, the longest of all being in the great spans of 550 feet built over the Ohio at Cincinnati in 1888 and at Louisville in 1893.

To recapitulate, the principles which should control the arrangement of a simple truss are the following: first, the panel system whereby the inclined members in the webbing are kept at approximately the same angle; second, the use of counterbraces to prevent distortion under a rolling load; third, that compression members should be made as short as possible; fourth, that a single system of webbing is always preferable, and that intermediate floor beams may be supported when necessary by the use of independent verticals; and fifth, that the form of the truss should be such that the stresses in members of the same kind may be approximately equal.

In addition to the references at the close of Arts. 1 and 2 the following may be noted as treating of the development of trusses: Bridge Superstructure, a committee report in Transactions American Society of Civil Engineers, 1878, Vol. VII, pp. 339–368; The Annual Address by JOSEPH M. WILSON, President of the Engineers' Club of Philadelphia, in the Proceedings of that Club for 1889, Vol. VII, pp. 65–104; The Evolution of the Modern Bridge by CHARLES D. JAMESON in Popular Science Monthly, Feb. 1890, pp. 461–481; and The Evolution of Bridge Trusses by MANSFIELD MERRIMAN in The Railway Age for May 19, 1893, Vol. XVIII, pp. 391–393. Bridge development with respect to other ideas than mere arrangement of truss members embraces the subjects of cantilever, continuous, suspension, and arched structures, which are reserved for discussion in Part IV.

## ART. 4. MATERIALS USED IN BRIDGES.

Prior to 1840 wood was the material used in this country for bridge construction. Great skill in carpentry was developed to devise the joints, mortises, keys, and other connections, although little was known regarding the strength of the timber or the rational principles of designing the proportions of the parts. The Howe truss combined the use of wood and iron in a most simple and successful manner, wrought-iron adjustable tie rods being used for the vertical members of the web, while the wooden diagonals butted against cast-iron angle blocks. In the original Pratt truss cast-iron joint connections were also employed, through which the wrought-iron diagonal ties passed. The first bridges wholly in iron had the compression members of cast iron and the tension members of wrought iron, this being, as WHIPPLE advocated, the best theoretic combination, since cast iron is high in compressive and low in tensile strength. Wrought iron, moreover, was high in price, and could then scarcely be obtained except in the form of simple rods.

Bridges of cast and wrought iron were built extensively until about 1870 and many of short span since that year, but the numerous failures of the cast-iron parts led to the gradual substitution of wrought iron. Probably the first bridge in which both compression and tension members were made of wrought iron was that erected on the Lehigh Valley Railroad at Mauch Chunk in 1863, but in this cast-iron joint connections were used. Gradually but surely wrought iron displaced cast iron, both for truss members and for joint details, so that by the centennial year 1876 cast iron was regarded as a material wholly inappropriate for use in bridge structures for railroad purposes, and the period of wrought-iron bridge development was at its height. But about this time steel began to be introduced.

The first extensive application of steel was in 1873 in the arches of the great St. Louis bridge, and later it was used in the trusses of the Brooklyn suspension bridge. In ordinary trusses it was at first employed in the form of eyebars for tension members, and then for the webs of floor beams. But improvements in the methods of manufacture soon followed, so that now angles, beams, channels, and other shapes of medium or mild steel are easily obtainable in the market at the same price as those of wrought iron. This steel closely resembles wrought iron, but its strength is about ten or fifteen per cent higher, and hence it is now used in bridge construction as frequently as wrought iron. The word steel is somewhat indefinite, on account of the many varieties which are produced for different purposes, but the properties of the structural steel required for bridges will be found set forth in the specifications given in later chapters.

The average life of iron railroad bridges is probably not far from twenty years, although under heavy traffic many are replaced after fewer years of service. They are liable to rust and corrosion from atmospheric influences and from the gases from the locomotives, while rivets and other connections are worn and loosened under the frequently repeated stresses and shocks. Bridges built twenty years ago are now generally unable to carry the heavier rolling stock with a proper margin of security. Hence an iron structure with respect to durability cannot compete with stone, and accordingly many roads are replacing short spans by arches of stone. The cheapness of iron and steel, however, generally renders metallic structures more economical in spite of their shorter life, and of course for long spans no other materials are available.

Some interesting notes by SQUIRE WHIPPLE on early iron bridges will be found in The Railroad Gazette, April 19, 1889. A historical paper on steel manufacture in America by W. F. DURFEE is given in The Popular Science Monthly, Oct. 1891, pp. 729–749. See also COOPER'S American Railroad Bridges, originally published in Transactions American Society of Civil Engineers for 1889, Vol. XXI, pp. 1–28.

## ART. 5. JOINT CONNECTIONS.

The members of the early wooden bridges, such as the Burr truss and the Long truss, were connected together by means of joints devised especially for timber structures. The fish and scarf joints employed in the chords are still used in the Howe truss and in other wooden constructions, but most of the special devices of shoulders, mortises, and keys now exist only in a few isolated examples.

The combination trusses which next followed, like the Howe and Pratt, employed the method of screw connections to join the webbing to the chords. In the Howe truss the several pieces of the chords were bolted together laterally, and connected longitudinally by fish joints so as to form one continuous member, but the web struts butted against angle blocks and were held in place by screwing up the vertical iron tie rods. The Pratt truss in its early forms had wooden chords upon which was placed at each panel point a cast iron joint block, and through this passed the diagonal iron ties which terminated in screws and nuts by which the whole was held in place. This method was also extensively used in the Pratt trusses built of cast and wrought iron, and many special forms of screw connections were devised and employed. In general, however, most of these screw joints have gone out of use, on account of the greater cheapness and reliability of the methods of riveted and pin connections.

The riveted system of connections is the prevailing method of construction in Europe, but in this country it is mostly limited to plate girders and to lattice trusses less than 200 feet in span. In this system the chords are formed of angles, or channels, and plates, riveted together, with splice joints so as to make them practically continuous from end to end; and the web members are connected to the chords by rivets, either directly or by means of special plates riveted to both. The first riveted bridges in this country were erected on the New York Central Railroad about 1860, and the system has proved very serviceable there and elsewhere.

The pin system of connections is the one which has been most used and which has generally been regarded with the most favor by American engineers. At each panel point a pin, or round bar, passes through holes in the chord or web members and serves to transfer the longitudinal stresses from one member to another by means of the shearing and bending stresses generated in it. Some of the early bridges built by WHIPPLE had pins which passed through looped eyes in the tension members, but the first bridge which was pin-connected throughout was erected by J. W. MURPHY in 1859 on the Lehigh Valley Railroad at Phillipsburg, N. J. Wide forged eyebars in connection with pins were first used in 1861 by J. H. LINVILLE on the Pennsylvania Railroad. The system then rapidly spread on account of ease of erection, and thousands of pin-connected bridges are now in service.

Much might be said in comparison of the riveted and pin systems. Advocates of the former claim that it makes a stiffer structure and one less liable to accident from the failure of a single member. Advocates of the latter say that the stresses in the pin system are more determinate and that better workmanship is secured. But under present conditions the question of economy seems the controlling factor. A long span cannot be built as cheaply by the riveted system as by the other, and a short or medium span can sometimes be built more cheaply. Under proper specifications a good bridge can be designed and erected on either plan, and the item of cost will usually determine the decision. The riveted system generally requires a little more material than the pin system, and the latter requires more skilled workmanship. High prices for iron and labor were favorable to the development of the pin system, and as these become lower the riveted system comes more and more into use.

The literature noted in the preceding articles contains much information regarding the various methods of joint connections. Further reference is made to the works named in the following pages, and also to a series of articles on Expired Bridge Patents by F. B. BROCK, in Engineering News during 1882 and 1883.

## ART 6. LITERATURE OF BRIDGE DESIGN.

The computation of stresses in the principal members of a bridge truss is the least part of the work of design, and hence books treating mainly on stresses are not noted in the following list. Bridge design includes of course the economic principles regarding the form of the truss, some of which have been mentioned in Art. 3, but more specifically it is the science of details, that is, the proportioning of the members, the floor, the joints, and of all the splices, reinforcing plates, rivets, pins, and other parts which make up the structure. The list of books below includes such as treat wholly or in part of these topics, together with a few of historical and descriptive character. Although not complete, it is believed that it gives the works on Bridge Design most important for a college library and for the use of American students of bridge design. The list is arranged chronologically according to the date of the first editions.

POPE, T. A Treatise on Bridge Architecture. New York, 1811. This contains 196 pages of descriptions of early bridges, while the remainder is devoted to the author's "patent flying pendant lever bridge."

WHIPPLE, S. A Work on Bridge Building. Utica, N. Y., 1847, pp. 120 and 10 plates. The edition of 1869 contains also 128 pages of notes (printed by the author's own hands) explanatory of the original work. See Art. 2.

DUGGAN, G. Stone, Iron and Wood Bridges of United States Railroads. New York, 1850. Consists mostly of drawings, with brief descriptive notes.

HAUPT, H. General Theory of Bridge Construction. New York, 1851, pp. 268 with 16 plates, giving examples of railroad bridges.

VOSE, G. L. Handbook of Railroad Construction. Boston, 1857, pp. 480. Contains 109 pages on wood, iron and stone bridges.

HUMBER, W. Cast and Wrought Iron Bridge Construction. London, 1864, two volumes, with 80 plates, mostly descriptive of English bridges.

HEINZERLING, F. Die Brücken in Eisen. Leipzig, 1870, pp. 515. A historical and descriptive work of bridge development in all countries. Also Die Brücken der Gegenwart. Leipzig, 1884, pp. 754 with 60 plates.

MERRILL, W. E. Iron Truss Bridges for Railroads. New York, 1870, pp. 130. A comparison of seven kinds of trusses with respect to theoretic economy.

BOLLER, A. P. Construction of Iron Highway Bridges. New York, 1876, pp. 144. Although written for the use of town committees this book has been of much value to young engineering students.

DU BOIS, A. J. Strains in Framed Structures. New York, 1883, pp. 390 with 27 plates. This devotes 124 pages to design, and gives the complete design of a pin-connected bridge. The edition of 1890 has 209 pages on design and erection.

WADDELL, J. A. L. Designing of Ordinary Iron Highway Bridges. New York, 1884, pp. 244 and 7 plates. A book which has done much to improve the design of highway structures.

BENDER, C. Principles of Economy in the Design of Metallic Bridges. New York, 1885, pp. 195 with 9 plates. This does not treat of details, but gives critical theoretic comparisons of different forms of trusses.

RICKER, N. C. Construction of Trussed Roofs. New York, 1885, pp. 158. Mainly deals with stresses, but has two chapters on dimensioning and details.

BURR, W. H. Stresses in Bridge and Roof Trusses. New York, 1886, pp. 454 with 12 plates. Devotes 112 pages to details and to the design of a railway bridge.

SCHÄFFER, T., and SONNE, E. Der Brückenbau (Vol. II of Handbuch der Ingenieur Wissenschaften). Leipzig, 1886–90, pp. 1812 with 77 plates.

HIROI, I. Plate Girder Construction. New York, 1888, pp. 94. Gives the design and estimate for a span of 50 feet.

MORANDIÈRE, R. Traité de la Construction des Ponts et Viaducs. Paris, 1888, pp. 1891 with 332 large plates.

COOPER, T. American Railroad Bridges. New York, 1890, pp. 58 with 27 plates. A historical and descriptive work of special value.

JOHNSON, BRYAN and TURNEAURE. Modern Framed Structures. New York, 1893, pp. 517 with 37 plates. This gives 238 pages on details with designs of several bridge structures.

A number of monographs on large bridges have also been issued in book form which are of special value to advanced students and engineers. Among these are The Quincy Bridge by T. C. CLARKE, 1869, The Kansas City Bridge by O. CHANUTE, 1870, The Omaha Bridge, The Cairo Bridge and others by G. S. MORISON, 1889–92, and The Thames River Bridge by A. P. BOLLER, 1891, and there are several others on suspension and arched bridges which will be mentioned in Part IV.

The number of articles on bridge design published in technical journals and transactions of engineering societies is so large that the attempt to give a list of them will not here be made. The Transactions of the American Society of Civil Engineers contain many papers both descriptive and critical. Of the latter class may be noted Specifications for the Strength of Iron Bridges by JOSEPH M. WILSON in 1886, Vol. XV, pp. 410–490, and Some Disputed Points in Railway Bridge Designing by J. A. L. WADDELL in 1892, Vol. XXVI, pp. 77–282. The discussions of engineers on these papers will be of especial value to students in familiarizing them with the leading lines of thought on questions of economic construction.

The volumes of Engineering News, The Railroad Gazette, The Engineering Record, and other technical periodicals contain numerous articles both theoretical and descriptive on bridge design, and some of these will be mentioned in the following chapters. The Descriptive Index of Current Engineering Literature, published by the Association of Engineering Societies, gives many pages of titles of such articles, with brief notes of their contents, and this should be at the hand of every student who desires to become well informed on the progress of bridge development. But it cannot be too strongly urged upon the student to form the habit of making his own catalogue of articles and of giving under each title his own synopsis of its contents and conclusions. By so doing he acquires a training in technical literary work which will be of the greatest value in promoting his professional advancement.

# CHAPTER II.

## PRINCIPLES OF ECONOMIC DESIGN.

### ART. 7. NUMBER OF PIERS AND SPANS.

When a bridge is to be built across a river, one of the first considerations is that regarding the number of spans. This question is to be decided by the principle that the total cost of the substructure and superstructure shall be a minimum, and the local circumstances of each particular case must be carefully regarded in making the preliminary investigations. In any event there will be two land abutments, and if the distance between these be short no intermediate piers are advisable. Yet it is seen even here that if piers could be erected without any expense it would be best to use them. Thus the relative cost of piers and their connecting spans determines the number of piers and spans which can be most economically built between the two abutments.

An old rule for this case states that the cost of the superstructure must equal the cost of the substructure in order that the cost of the whole may be a minimum. The cost of piers is to be determined by careful surveys and estimates for various locations along the line, while the cost of spans of different length may be approximately ascertained by consulting builders. A comparison of the different possible arrangements determines the most economic plan which sometimes agrees well with this rule.

The cost of ordinary bridges is closely proportional to their weights. If $l$ be the length of span, the formula $W = al + bl^2$

gives a good approximation to the weight (Part I, Art. 45), $a$ and $b$ being constants for the same type of truss. In this $al$ represents the weight of the track and floor system, while $bl^2$ represents the weight of the main trusses and lateral bracing. For example, the total weight of iron in pounds in a single-track railroad lattice bridge (not including cross-ties and rails) is about $200l + 7l^2$, if $l$ be the span in feet, while that of a pin-connected bridge is $350l + 5l^2$.

If the cost of intermediate piers is about equal and they be spaced at equal distances apart, the following investigation will give the economic number of spans. Let $l$ be the total distance between end abutments, $x$ the number of spans, and hence $x - 1$ the number of piers, $m$ the cost of the two abutments, $n$ the cost of each pier, and $p$ the cost per pound of the bridge superstructure. The weight of the $x$ spans, each of length $\frac{l}{x}$, is then $x\left(a\frac{l}{x} + b\frac{l^2}{x^2}\right)$, and the total cost of the work is

$$C = m + n(x - 1) + p\left(al + \frac{bl^2}{x}\right).$$

This will be a minimum when the first derivative of $C$ with respect to $x$ becomes zero, and this gives $n = pb\frac{l^2}{x^2}$, which shows that the cost of one of the intermediate piers should equal the cost of the main and lateral trusses of one of the spans. Or, $x = \sqrt{\frac{pbl^2}{n}}$ gives the economic number of spans. For example, if $l = 1000$ feet, $a = 350$ and $b = 5$ for pin-connected spans, and $p = 5$ cents per pound, then for $n =$ \$5000, the most economic number of spans is $x = 7$, and the total cost is \$83 200, exclusive of abutments. Here the cost of the intermediate piers is \$30 000 and that of the seven spans is \$53 200, which indicates that the old rule may sometimes be at fault. Again, if the cost of a pier be $n =$ \$10 000, the economic number of spans is $x = 5$, which gives \$40 000 for the piers and \$87 500 for the superstructure.

## ART. 8. CHOICE OF KIND OF BRIDGE.

Whether the bridge span is to be deck or through will be determined in each case by the local conditions, among which the grades of the approaches are controlling factors. A deck span is usually cheaper than a through one, since the width of the bridge may be less and something is also saved on abutments and piers, and should hence be chosen if the approaches allow it and proper waterway can be secured beneath it.

The width of the bridge between trusses is determined by the amount of traffic. For a single track railroad this width for a through bridge is taken as 14 or 15 feet in the clear, while for a deck bridge 10 or 12 feet between centers of trusses is usually enough for short or medium spans.

The cost of the bridge is a material factor in determining the kind which is to be erected, and the problem of selection is hence a very complicated one. For railroads experience has led to the conclusion that at present the best results both as to stability and economy are obtained by using solid rolled beams for short spans up to 15 or 20 feet, plate girders for spans from 15 to 90 feet, riveted lattice trusses for spans from 50 to 150 feet, and pin-connected trusses for spans over 100 feet. It will be observed that these figures overlap each other, indicating that there is no distinct line of demarcation between the lengths of spans of the different classes.

The particular kind of truss is not usually stated in the specifications, this being left to the bidders who often may present plans which differ materially in general appearance. If all these plans conform to the specifications the contract is awarded to the lowest responsible bidder. The choice of the kind of truss is hence usually made by the sellers rather than by the buyers of bridges, but it may be that the estimate of the lowest bidder is sometimes influenced by the form of truss adopted.

The discussion in Art. 3 gives only the general economic conditions which determine the form of truss. The depth of the truss is to be selected so as not only to secure proper head-way and afford opportunity for cross bracing, but also so as to give the least amount of material; this question of economic depth is investigated in Art. 10. The number of panels should be odd rather than even for best economy, and should be such that the panel lengths, or distances between floor beams, may range from 12 to 24 feet. Probably the best panel length, as far as the floor system is concerned, is that which renders the weight of a floor beam about equal to that of the stringers in one panel.

Æsthetic considerations should not be overlooked in choosing the kind of bridge, and the old maxim that strength, beauty, and economy go together contains some truth. The parabola is a line of beauty, and trusses with parabolic chords are among those which now seem to be leading to the highest degree of economy.

## ART. 9. THEORETIC COMPARISONS OF TRUSSES.

Approximate economic comparisons of trusses of different forms may be made by comparing the theoretic amounts of material, the material in any member being taken as proportional to the product of its maximum stress by its length. Investigations of this kind were first made by WHIPPLE in 1847, and have since proved of much value especially in studying the question of economic proportions. The following is a simple example illustrating the method.

Let a Pratt truss and a triangular lattice truss, each having nine panels on the lower chord, have the same span and depth. Figs. 6 and 7 show one half of each truss. Let the dead load per panel point be $w$, and the live load be three times as large, or $3w$. Let the depth of each truss be $h$, and the panel length be $p$; then the tangent and secant of the angle which a diagonal makes with the vertical are $\tan\theta = \frac{p}{h}$ and $\sec\theta = \frac{\sqrt{h^2+p^2}}{h}$. By the methods of Part I the maximum stresses in each member for the given loads are computed, and

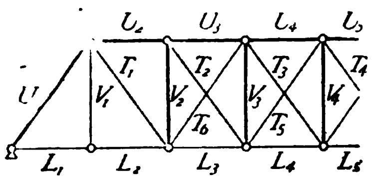


FIG. 6.

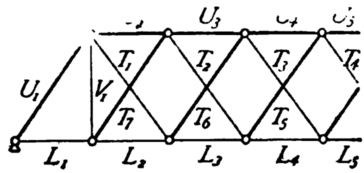


FIG. 7.

recorded in the second and fourth columns of the table in terms of $\tan\theta$ and $\sec\theta$. Replacing $\tan\theta$ and $\sec\theta$ by their values, and multiplying each stress by the length of the corresponding member, the products in the third and fifth columns are obtained, each of which is proportional to amount of material if the working unit stresses be the same throughout. As the object is to compare the material for one half of each truss the stresses for $L_5$ and $U_5$ are multiplied by $\frac{1}{2}p$ instead of $p$. The sums of these products, given at the foot of the table, thus approximately represent the relative amounts of material in the two trusses, neglecting joint connections, lateral bracing and floor system, which may be supposed to be the same for both.

These sums show that the lattice truss is theoretically the more economical in respect to material. For a particular example let the span be 162 feet or $p = 18$ feet. Then these sums give the following comparisons for different depths of truss:

| | $h$ = 18 feet. | $h$ = 24 feet. | $h$ = 30 feet. | $h$ = 36 feet. |
|---|---|---|---|---|
| Pratt............ | $6432w$ | $5563w$ | $5207w$ | $5109w$ |
| Lattice.......... | $6096w$ | $5132w$ | $4682w$ | $4488w$ |
| Per cent difference | 5.5 | 8.4 | 11.2 | 13.6 |

| Member. | Pratt Truss. | | Lattice Truss. | |
|---|---|---|---|---|
| | Stress. | Stress × Length. | Stress. | Stress × Length. |
| $L_1$ | $+16w\tan\theta$ | $16w\frac{p^2}{h}$ | $+16w\tan\theta$ | $16w\frac{p^2}{h}$ |
| $L_2$ | $+16w\tan\theta$ | $16w\frac{p^2}{h}$ | $+20w\tan\theta$ | $20w\frac{p^2}{h}$ |
| $L_3$ | $+28w\tan\theta$ | $28w\frac{p^2}{h}$ | $+32w\tan\theta$ | $32w\frac{p^2}{h}$ |
| $L_4$ | $+36w\tan\theta$ | $36w\frac{p^2}{h}$ | $+36w\tan\theta$ | $36w\frac{p^2}{h}$ |
| $L_5$ | $+40w\tan\theta$ | $20w\frac{p^2}{h}$ | $+40w\tan\theta$ | $20w\frac{p^2}{h}$ |
| $T_1$ | $+12\frac{1}{2}w\sec\theta$ | $12\frac{1}{2}w\left(h+\frac{p^2}{h}\right)$ | $+7\frac{1}{2}w\sec\theta$ | $7\frac{1}{2}w\left(h+\frac{p^2}{h}\right)$ |
| $T_2$ | $+9w\sec\theta$ | $9w\left(h+\frac{p^2}{h}\right)$ | $+5w\sec\theta$ | $5w\left(h+\frac{p^2}{h}\right)$ |
| $T_3$ | $+6w\sec\theta$ | $6w\left(h+\frac{p^2}{h}\right)$ | $+4w\sec\theta$ | $4w\left(h+\frac{p^2}{h}\right)$ |
| $T_4$ | $+3\frac{1}{2}w\sec\theta$ | $3\frac{1}{2}w\left(h+\frac{p^2}{h}\right)$ | $+2w\sec\theta$ | $2w\left(h+\frac{p^2}{h}\right)$ |
| $T_5$ | $+w\sec\theta$ | $w\left(h+\frac{p^2}{h}\right)$ | $-2w\sec\theta$ | $2w\left(h+\frac{p^2}{h}\right)$ |
| $T_6$ | 0 | 0 | $-4w\sec\theta$ | $4w\left(h+\frac{p^2}{h}\right)$ |
| $T_7$ | | | $-5w\sec\theta$ | $5w\left(h+\frac{p^2}{h}\right)$ |
| $V_1$ | $+4w$ | $4wh$ | $+8w$ | $8wh$ |
| $V_2$ | $-9w$ | $9wh$ | | |
| $V_3$ | $-6w$ | $6wh$ | | |
| $V_4$ | $-3\frac{1}{2}w$ | $3\frac{1}{2}wh$ | | |
| $U_1$ | $-16w\sec\theta$ | $16w\left(h+\frac{p^2}{h}\right)$ | $-16w\sec\theta$ | $16w\left(h+\frac{p^2}{h}\right)$ |
| $U_2$ | $-28w\tan\theta$ | $28w\frac{p^2}{h}$ | $-24w\tan\theta$ | $24w\frac{p^2}{h}$ |
| $U_3$ | $-36w\tan\theta$ | $36w\frac{p^2}{h}$ | $-32w\tan\theta$ | $32w\frac{p^2}{h}$ |
| $U_4$ | $-40w\tan\theta$ | $40w\frac{p^2}{h}$ | $-40w\tan\theta$ | $40w\frac{p^2}{h}$ |
| $U_5$ | $-40w\tan\theta$ | $20w\frac{p^2}{h}$ | $-40w\tan\theta$ | $20w\frac{p^2}{h}$ |
| | Sum $=\left(70h+287\frac{1}{2}\frac{p^2}{h}\right)w$ | | Sum $=\left(53\frac{1}{2}h+285\frac{1}{2}\frac{p^2}{h}\right)w$ | |

It is seen that the Pratt truss requires from 5.5 to 13.6 per cent more material than the lattice for the different depths considered in this particular case. These percentages, however, would be considerably reduced if the unit stresses were taken less for compression than for tensile members, and if the range of stresses were considered. The lattice truss has diagonal compression members also, whose working unit stresses should be less than for the shorter verticals of the Pratt truss. But to include all these elements in a theoretic comparison would lead to great complexity, and it is in fact only by making actual designs that results completely reliable can be obtained. But many useful conclusions can be derived from such investigations, one of which will be given in the next article, and others may be seen in BENDER'S work quoted in Art. 6.

## ART. 10. ECONOMIC DEPTH.

The economic depth of a girder or truss is that which renders its weight a minimum. Such a depth exists by virtue of the facts that the chord material decreases and the web material increases as the depth is increased. For a plate girder it is a rough general rule that the economic depth obtains when the weight of the flanges is equal to the weight of the web. To show this it must be borne in mind that the thickness of the web plate is practically constant for a girder of short span, being rarely greater than $\frac{5}{8}$ nor less than $\frac{3}{8}$ inch. The material in the web hence varies as $a \times h$, and that in the flanges as $b \div h$, where $a$ and $b$ are constants depending on the span loads and working unit stresses. The total material may then be represented by $a \times h + b \div h$, which is a minimum when the two terms are equal, that is when the flange weight equals the web weight, or, more strictly, when the cost of the flanges equals the cost of the web. This problem is more fully discussed in connection with an actual design in Art. 47, and a formula for economic depth is given in Art. 125.

For a truss, however, the weight of the horizontal chords should be less than that of the webbing to give the greatest economy. For example, take the Pratt truss discussed in the last Article whose depth is $h$ and panel length $p$. The total theoretic amount of material in one half the truss was found to be $70wh + 287\frac{5}{8}wp^2 \div h$, of which $70wh + 47\frac{5}{8}wp^2 \div h$ is in the webbing including the two end struts, and $240wp^2 \div h$ in the chords. The theoretic economic depth $h$ is found by differentiating the expression for the total amount of material and equating the derivative to zero, which gives $h = 2.03p$. For this depth the web material reduces to $165.5wp$ and the chord material to $118.2wp$, the former being 40 per cent the greater. Practically, however, the economic depth should be considerably less than $2.03p$, since the investigation does not include the increase of material due to strengthening the compression members of the web.

A general theoretic discussion for any given type of truss may be undertaken by computing the stresses due to uniform panel loads in terms of the number of panels. Let $p$ be the panel length, $h$ the depth of the truss, and $n$ the number of panels, which is taken as even. Let the truss be of the Whipple type (Fig. 3) and be loaded on the lower chord with $w$ at each panel point. The stress in the inclined end strut is $\frac{1}{2}(n - 1)w \sec \theta$, and the product of stress by length is $\frac{1}{2}(n - 1)\left(h + \frac{p^2}{h}\right)w$. Finding these products for each member, and adding, there results $n(n^2 - 1)\frac{wp^2}{6h}$ for the chords and $(n^2 - 2n + 16)\frac{wh}{4} + (n^2 - 3n + 4)\frac{wp^2}{2h}$ for the webbing. The minimum of the sum of these is determined by the usual method, and the economic depth is found to be given by $\frac{h}{p} = \sqrt{\frac{2(n^3 + 3n^2 - 10n + 12)}{3(n^2 - 2n + 16)}}$. Taking the panel length as a constant, the following comparison for different numbers of panels can now be made:

| For.................. | $n = 6$ | $n = 10$ | $n = 14$ | $n = 18$ |
|---|---|---|---|---|
| $h \div p$.............. = | 2.14 | 2.90 | 3.40 | 3.81 |
| Span $\div$ depth....... = | 2.8 | 3.4 | 4.1 | 4.7 |
| Web material....... | 26.5 | 82.4 | 178.9 | $324.8wp$ |
| Chord material...... | 16.3 | 56.9 | 133.9 | $254.3wp$ |
| Per cent difference.. | 63 | 44 | 34 | 28 |

This discussion shows that the ratio of economic depth to panel length increases, while the ratio of depth to span decreases, with the length of span. The amount of web material is greater than that of the chords, but the percentage of excess in the former decreases as the span increases. These general conclusions are valuable, but the particular figures just given are materially modified in practice, since the actual economic depth is always found to be considerably less than the theoretical.

Finally, it should be noted that great exactness in regard to economic depth is not important, since a function changes slowly in the vicinity of a maximum or minimum, so that considerable variations in depth may be made without much increasing the total quantity of material. For example, in the above Whipple truss of ten panels, the economic depth is $h = 2.90p$, and for this and several other depths the comparative quantities of material are:

| $h \div p$......... | 2.5 | 2.7 | 2.9 | 3.1 | 3.3 |
|---|---|---|---|---|---|
| Web material. | 74.8 | 78.5 | 82.4 | 86.3 | $90.4wp$ |
| Chord material | 66.0 | 61.1 | 56.9 | 53.2 | $50.0wp$ |
| Total material | 140.8 | 139.6 | 139.3 | 139.5 | $140.4wp$ |
| Per cent excess | 1.1 | 0.2 | | 0.1 | 0.8 |

which indicates that the depth may vary 10 per cent from the economic depth while the increase in the total quantity of material is less than one per cent.

Instructive discussions of the question of theoretic economy of material and cost may be found in CREHORE'S Mechanics of the Girder, New York, 1886, as also in several of the works quoted in Art. 6.

### ART. 11. PRACTICAL CONSIDERATIONS.

The general conclusions of the preceding investigations have been confirmed by experience. It has been found that the depth of the Whipple truss should be about double the panel length for the best economy, and that the depth of the Pratt truss should be considerably greater, say $1\frac{1}{4}$ times, than the panel length. The depth of trusses has materially increased during the past twenty years, and probably the full economic development has not yet been attained.

The engineer who draws the specifications is primarily responsible both for the strength and security as well as for the economy of the structure. For, if improper working stresses are prescribed, or proper rules for stability are omitted, the builders, under the influence of competition, will present plans of structures lacking in security; or, if excessive and unusual requirements are made in the specifications, the plans presented will not be economical. At present there are so many specifications which may be called standard that it is not possible to go far astray in either of these directions, particularly for railroad bridges. For many highway structures, however, the specifications are very loosely drawn, and every year there are erected some bridges which are defective either in stability or economy. As a general rule economy demands a bridge of proper stability, and the proper degree of stability will be secured by structures of the best economic design.

The designer should, of course, strictly follow the specifications, yet in details and dimensions he has great liberty of choice. He should be well acquainted with the market sizes of materials and with the market prices. Variation from regular sizes always involves delay and extra cost. Uniformity of sizes is advantageous, since several things of one kind can be purchased or made more cheaply than if they are of different dimensions. Simplicity of connections should be studied not only with respect to strength, but also with regard to economy of manufacture.

In riveted work excessive nicety in the spacing of rivets should be avoided. If possible the pitch should be in even inches, that is either 2, 3, 4, 5 or 6 inches, especially when the rows are long, as in columns and the flanges of plate girders. It will be more economical still if the pitches can be reduced to two, 3 inches and 6 inches, but this is not so easy to attain and still maintain the proper uniform strength throughout.

In pin-connected work it will often be advantageous, particularly for short spans, if the pins are of uniform sizes, except perhaps those at the ends. As the strength of a pin depends more upon its resistance to transverse stresses than to shearing, it is often possible to insure that the prescribed unit-stresses shall not be exceeded by properly spacing the eyebars (Art. 54). Columns and lateral bracing must be arranged with due regard both to economy of shop work and to ease of erection. Field riveting should be reduced to a minimum, since it is more expensive and less satisfactory in regard to strength than shop riveting.

# CHAPTER III.

## TABLES AND STANDARDS.

### ART. 12. MANUFACTURERS' POCKET BOOKS.

The student of bridge design will find it absolutely necessary to have at hand one of the pocket books issued by the manufacturers of structural materials. There are several of these; the best known being those popularly called the Trenton, the Phœnix, the Pencoyd and the Carnegie pocket books. Of these the last two are the most complete and best adapted for work in bridge design. The Pencoyd book is entitled Wrought Iron and Steel in Construction by A. & P. ROBERTS & CO., Philadelphia. The Carnegie book is The Pocket Companion, edited by C. L. STROBEL, and issued by the Carnegie Steel & Iron Co., Limited, Pittsburg, Pa. The price of each of these is $2.00, but they can usually be obtained by college students in quantities at a lower figure.

These pocket books contain full tables of all the market shapes of iron and steel manufactured by the respective firms, stating weights, areas of sections, moments of inertia, radii of gyration and other constants. There are also given tables for rivet spacing, standard bolts, bridge pins, and others on the strength of materials, particularly concerning those used in bridge construction. By the use of these tables the computations necessary in bridge design are greatly shortened.

In the following pages are presented a few tables which are not all found in one of these pocket books, together with examples of their use. CARNEGIE'S Pocket Companion, edition of 1893, will be generally referred to and used in the computations of the designs in the four following chapters.

OSBORN'S Tables of Moments of Inertia for Column Sections and BUCHANAN'S Tables of Squares may be mentioned as books frequently used by computers in bridge offices, and which will also be serviceable to students.

## ART. 13. STANDARD LOADS FOR BRIDGES.

The word standard, as here used, means those loads which are widely used by the best authorities as necessary in order to design bridge structures which shall be sufficiently strong. Such loads, of course, are supposed to closely represent the maximum weights to which the structure is liable.

For highway bridges the following loads, which were recommended by J. A. L. WADDELL in 1884, have been very widely used, so that they approach nearer to a standard than any

| Span in Feet. | City Bridges. | Country Bridges. |
|---|---|---|
| 0 to 50 | 100 | 80 |
| 50 to 150 | 90 | 80 |
| 150 to 200 | 80 | 70 |
| 200 to 300 | 70 | 60 |
| 300 to 400 | 60 | 50 |

others. The quantities given are in pounds per square foot of floor surface, and are supposed to represent the weight of a crowd of people. Of course the floor beams and stringers must be proportioned for the weight of the heaviest wagons that are liable to cross the structure. The recent introduction of electric railways renders it advisable that most highway bridges should be designed with such in view, and reference is made to Arts. 66 and 132 for the loads of electric-motor cars.

For railroad bridges the "compromise standard system" of live loads recommended by WADDELL in 1893, after a discussion of the subject by many engineers, may be here noted as one which seems likely to be much used in the future. The typical consolidation locomotives are divided into seven classes, called T, U, V, etc., and the wheels in each class have the same spacing. Fig. 8 shows this spacing and the loads for class T,

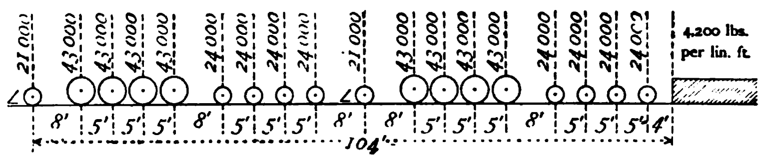



FIG. 8.

while the loads for the other classes are given in the table. The distance from the front pilot wheel to the beginning of the uniform load is in all cases 104 feet, and the loads are

| Class. | Load on Pilot Wheel. | Load on each Driving Wheel. | Load on each Wheel of Tender. | Total Weight of one Locomotive and Tender. | Uniform Load per Linear Foot. |
|---|---|---|---|---|---|
| Z | 15 000 | 25 000 | 18 000 | 187 000 | 3 000 |
| Y | 16 000 | 28 000 | 19 000 | 204 000 | 3 200 |
| X | 17 000 | 31 000 | 20 000 | 221 000 | 3 400 |
| W | 18 000 | 34 000 | 21 000 | 238 000 | 3 600 |
| V | 19 000 | 37 000 | 22 000 | 255 000 | 3 800 |
| U | 20 000 | 40 000 | 23 000 | 272 000 | 4 000 |
| T | 21 000 | 43 000 | 24 000 | 289 000 | 4 200 |

stated in pounds. Of course in designing a bridge the exact loads given in the specifications must be used, but it is hoped that engineers who write specifications will gradually abandon the awkward typical locomotives with wheels spaced apart at distances involving odd fractions of a foot, for surely this is straining at a precision unwarranted by actual conditions.

There can scarcely be said to be any figures for wind pressure and for impact allowances that can be called standard, and reference concerning them is made to the various specifications in later chapters of this book.

## ART. 14. RIVET PROPORTIONS.

The rivet proportions given in Fig. 9 are those adopted by the Pencoyd Bridge and Construction Company. For finished heads their diameter equals $1\frac{1}{2}$ times the diameter of the shank plus $\frac{1}{8}$ inch, and their depth equals 0.45 times their diameter; while for countersunk heads the depth is one half of the diameter of the shank, and their bevel 60 degrees.

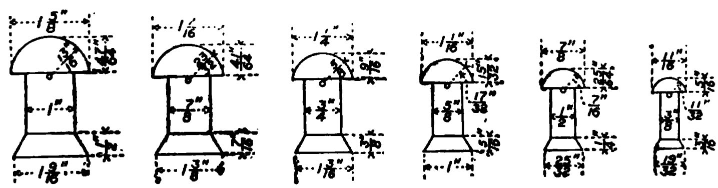


FIG. 9.

The proximity of rivets to the edges and ends of the shapes united is usually limited by the specifications, and the clearance between the rivet head and any adjacent surface must allow room for the riveting tool. The clearance should be about $\frac{3}{8}$ inch, but if necessary may be reduced. The minimum pitch of rivets is three diameters and the maximum is sixteen times the thinnest outside plate, but is not to exceed six inches except in special cases. Rivet tests show that the grip length should not exceed five diameters (see Engineering News, Dec. 6, 1890, Vol. XXIV, p. 500) for machine-driven rivets. It is sometimes specified that the grip length of field rivets (hand driven) will not be allowed to exceed three diameters.

The conventional signs to be used on drawings for different kinds of rivets are explained in Art. 21.

## ART. 15. RIVET SPACING IN ANGLES.

The following table gives the Pencoyd standard rivet spacing in angles. When angles are used for flanges of plate girders the rivet holes are put a little farther distant from the corner than when they are used for braces and stiffeners, since in the latter case it is desirable to have them as near the centre of gravity of the section as possible. Either a single or double row of rivets may be used as required by the stress to be transmitted. Fig. 10 shows the two cases, $l$ representing the

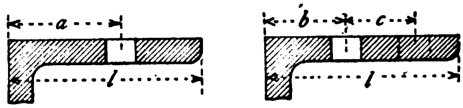



FIG. 10.

| Length of Angle Leg. | Spacing for Flanges. | | | Spacing for Braces. | | | Diameter of Rivets. |
|---|---|---|---|---|---|---|---|
| $l$ | $a$ | $b$ | $c$ | $a$ | $b$ | $c$ | |
| 7″ | | 2½″ | 3″ | | 2″ | 3½″ | ⅞″ |
| 6½ | | 2½ | 2½ | | 2 | 3 | ⅞″ |
| 6 | 3½″ | 2½ | 2¼ | | 2 | 2½ | ¾ or ⅞ |
| 5½ | 3½ | 2¼ | 2 | | 2 | 2¼ | ¾ or ⅞ |
| 5 | 3 | 2 | 1¾ | | 1¾ | 2 | ¾ or ⅞ |
| 4½ | 2½ | 2 | 1¼ | 2¼″ | 1¾ | 1½ | ¾ or ⅞ |
| 4 | 2¼ or 2½ | | | 2 | | | ¾ or ⅞ |
| 3½ | 2 | | | 1¾ | | | ¾ or ⅞ |
| 3 | 1¾ | | | 1⅝ | | | ¾ |
| 2¾ | 1⅝ | | | | | | ⅝ or ¾ |
| 2½ | 1⅜ | | | | | | ½ or ⅝ |
| 2¼ | 1¼ | | | | | | ½ or ⅝ |
| 2 | 1⅛ | | | | | | ½ |
| 1¾ | 1 | | | | | | ½ |

length of the angle leg, $a$ the distance from the corner to the pitch line of the single row of rivets, $b$ the distance from the corner to the pitch line of the first row for double riveting, and $c$ the distance between the pitch lines of the two rows. The preceding table gives the values of $a$, $b$, and $c$ corresponding to different values of $l$ for the angles commonly used in bridge structures, together with the proper diameters of the rivets.

## ART. 16. PIN PLATES AND RIVETS.

The diagram in Fig. 11 is constructed for the unit stresses and diameters of rivets there given. The diameters of the pins are laid off as abscissas, and the bearing values for the pins as ordinates, the linear bearing on the pins being marked on the lines radiating from the lower left-hand corner. The allowable stress for an 8-inch pin with a bearing of $1\frac{7}{8}$ inches is seen to be 180 000 pounds by following the ordinate for a diameter of 8 inches until it meets the radial line marked $1\frac{7}{8}$ inches, and reading off its value from the scale at the right.

The number of $\frac{7}{8}$-inch rivets in single shear is laid off at the top of the diagram, so that by following down any ordinate until the diagonal line (separating the two systems of horizontal and vertical ruling) is reached, the allowable shearing stress of the corresponding number of rivets may be read off by the scale on the right. Thus, the shearing value of 22 rivets is found to be 99 000 pounds. It may be added that the diagram as here printed is considerably reduced from the original size on which more precise readings could be made. Usually, however, it is not necessary to read closer than one thousand pounds. On the left side the number of $\frac{7}{8}$-inch rivets in bearing is laid off to such a scale that by following any horizontal line until it intersects a line radiating from the upper left-hand corner on which the thickness of plates, or the linear bearing of the rivets, is marked, the equivalent number of rivets in shear may be read off on the scale at the top. For instance, the bearing stress of 12 rivets in a $\frac{1}{2}$-inch plate is very nearly equal to the stress of 14 rivets in single shear.

By combining the two preceding operations the value of the bearing stress of 12 rivets in a $\frac{1}{2}$-inch plate may be obtained by following down from the point of intersection to

## PIN-PLATE AND RIVET DIAGRAM.

Unit stresses:—
Bearing of pin, 12 000 lbs. per square inch.
Bearing of rivets, 12 000 lbs. per square inch.
Shear of rivets, 7 500 lbs. per square inch.

$\frac{7}{8}$-inch rivets:—
Single shear, 4 510 lbs.
$\frac{5}{8}$-inch bearing, 3 940 lbs.
$\frac{7}{16}$-inch bearing, 4 590 lbs.

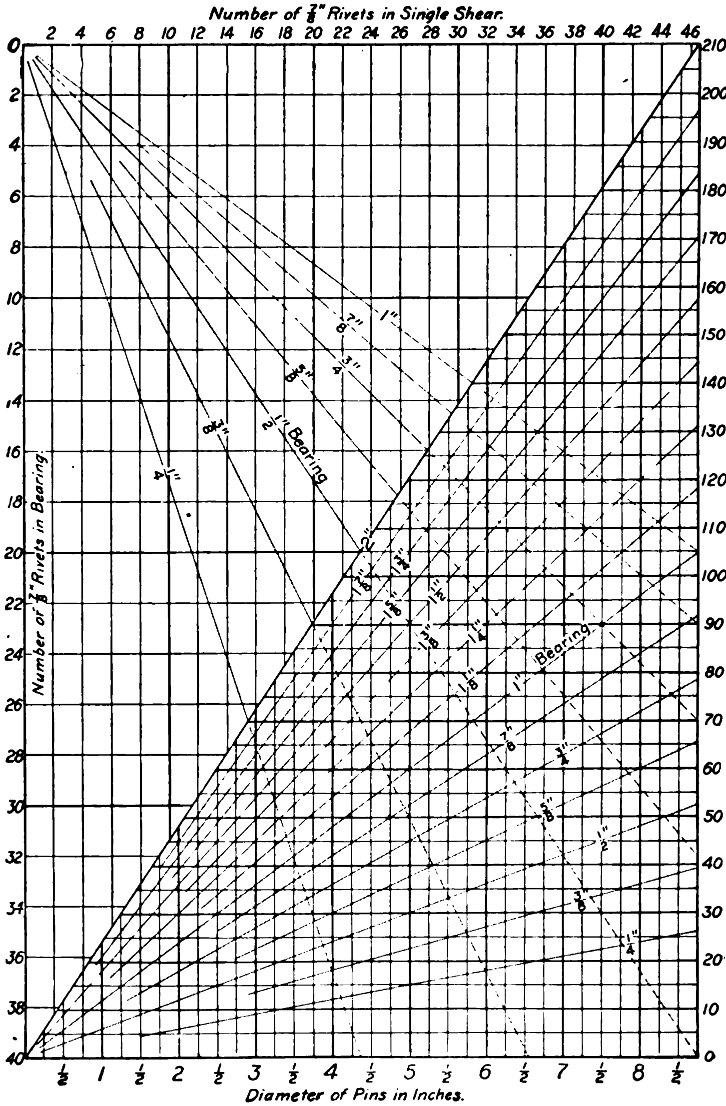



FIG. 11.

the diagonal line, and then reading the stress on the scale at the right, the value being 63 000 pounds. Furthermore all of the preceding operations may be combined. For example, the bearing value of a 6-inch pin with a bearing of $1\frac{1}{4}$ inches equals the shearing value of 20 rivets or the bearing value of 23 rivets in a $\frac{5}{8}$-inch plate. The upper radiating lines go beyond the diagonal in order to extend the limits of the diagram.

To further illustrate the use of the diagram let the pin plates and rivets shown in Fig. 12 be investigated. Fig. 12 shows the end of an upper chord copied from the plans of a through railway bridge constructed several years ago.

For the given unit stresses the bearing of a rivet in a $\frac{5}{8}$-inch plate is less, and that in a $\frac{7}{16}$-inch plate is greater than the value for single shear, therefore the number of rivets in the inside pin plate depends on their bearing, while the number of rivets in the three outside plates depends upon single shear. The stress transmitted to the $\frac{5}{8}$-inch plate from the $6\frac{5}{8}$-inch pin is shown by the diagram to be 29 000 pounds and requires 8 rivets in bearing. The elevation of the chord in Fig. 12 shows 15 rivets passing through this pin plate. The outside $\frac{1}{2}$-inch plate needs 9 rivets in single shear, while 11 are used. The rivets passing through the second outer pin plate must transmit the stress from both the $\frac{1}{2}$-inch plates. Following the ordinate at $6\frac{5}{8}$ inches to the line of 1-inch bearing, then across to the diagonal, the number of rivets in shear is found to be 17, while the number inserted is 24. The bearing of the pin on the three outer plates requires 28 rivets in single shear to pass through the $\frac{5}{8}$-inch plate, while it is seen that 31 are employed. It is observed therefore that so far as the pin plates themselves are concerned they have a sufficient number of rivets.

The next problem is to find what total stress can be transmitted by all the pin plates to the shapes composing the chord. Four of the rivets in the upper angle and three in the lower are in double shear, and the stress they can carry depends on their ⅜-inch bearing in the inner pin plate and on

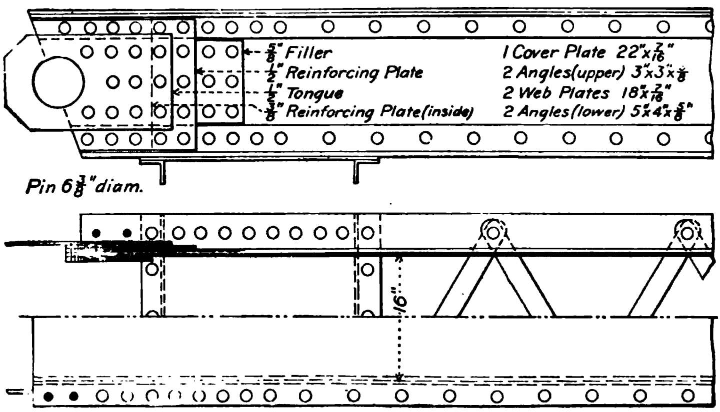



FIG. 12.

single shear outside of the web. Eight additional rivets in the web are in double shear, and the stress they can transmit to the web depends on their $\frac{7}{16}$-inch bearing in the web. All the rest depend on single shear. On summing up the total stress is obtained, as follows:

| | | |
|---|---|---|
| Bearing stress of 7 rivets in ⅜-inch plates...... | 28 000 | pounds |
| Bearing stress of 8 rivets in $\frac{7}{16}$-inch plates.... | 37 000 | " |
| Shearing stress of 23 rivets.................. | 104 000 | " |
| Total stress............................ | 169 000 | pounds |

This amount exceeds 153 000 pounds, which is the bearing stress of the pin on the four pin plates. The above result was obtained on the supposition that it makes no difference how the stress received from the pin plates is distributed among the shapes composing the chord. It is apparent that the design was made on this supposition.

Now let the arrangement of the rivets be tested on the basis that the stresses should be distributed in proportion to the areas of the shapes. The total stress of one half of the chord equals 153 000 plus the bearing of the pin on the web, making a total of 186 500 pounds, divided as follows:

| | | |
|---|---|---|
| ½ cover plate.... .................. | 44 800 | pounds |
| 1 upper angle ..................... | 19 600 | " |
| 1 web plate........ .............. | 73 400 | " |
| 1 lower angle........................ | 48 700 | " |
| | 186 500 | pounds |

The stress in the half cover plate and the upper angle, amounting to 64 400 pounds, requires 17 rivets in ⅝-inch bearing to transmit the stress into the angle, while only 6 were used. The five rivets in the bottom angle can only transmit 28 700 pounds, which is 20 000 pounds less than required. The number used in the web is much in excess of the theoretic demand. This investigation shows that some of the pin plates should have been materially lengthened, the smaller pitch of the rivets in the angle being maintained for nearly double the distance shown on the drawing.

For an example in the design of pin plates and their rivets see Art. 58.

## ART. 17. PROPERTIES OF CHANNELS.

The following table gives the dimensions, areas of cross-sections, and moments of inertia for some of the Pencoyd channels. In general these elements are stated only for the minimum sections rolled for each size, while in the last column are found the maximum sections that may be obtained. For example the first 8-inch channel whose properties are given has an area of 4.25 square inches, but other sizes with greater thicknesses of web and widths of flanges may be obtained up to a maximum of 8.05 square inches. More complete tables of channels may be found in the Pencoyd and other hand books.

As an example of its use suppose that a post is to be formed of two 9-inch channels of the first kind given, the flanges being turned inward (as seen on Plate IV). It is required to find the distance from back to back of channels in order that the moments of inertia about the two rectangular axes through the centre of the section may be equal, thus rendering the column of equal strength against lateral flexure in

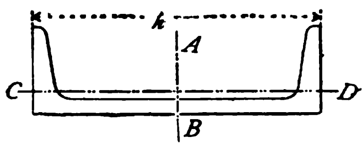



FIG. 13.

| Size of Channel. $k$ | Width of Flange. | Thickness of Web. | Area of Section. | Axis normal to Web. Moment of Inertia. | Axis normal to Web. Radius of Gyration. | Axis parallel to Web. Moment of Inertia. | Axis parallel to Web. Radius of Gyration. | Axis parallel to Web. Distance from Base to Axis. | Areas of Maximum Sections obtainable. |
|---|---|---|---|---|---|---|---|---|---|
| 15 | 4.125 | 0.75 | 16.21 | 457.03 | 5.32 | 18.39 | 1.07 | 0.84 | 16.21 |
| 15 | 3.875 | 0.375 | 10.44 | 351.56 | 5.79 | 13.08 | 1.12 | 1.00 | 16.21 |
| 15 | 4.00 | 0.625 | 14.86 | 451.51 | 5.51 | 19.05 | 1.13 | 0.95 | 20.45 |
| 12 | 2.937 | 0.406 | 8.83 | 182.71 | 4.55 | 7.42 | 0.92 | 0.71 | 16.00 |
| 12 | 2.609 | 0.281 | 5.94 | 123.71 | 4.56 | 3.22 | 0.74 | 0.62 | 10.15 |
| 10 | 2.594 | 0.281 | 5.99 | 92.08 | 3.92 | 4.29 | 0.84 | 0.75 | 10.60 |
| 10 | 2.375 | 0.25 | 4.89 | 73.91 | 3.89 | 2.33 | 0.69 | 0.64 | 8.65 |
| 9 | 2.437 | 0.312 | 5.40 | 64.34 | 3.45 | 2.47 | 0.68 | 0.67 | 9.20 |
| 9 | 2.141 | 0.234 | 3.72 | 43.65 | 3.43 | 1.31 | 0.59 | 0.55 | 6.10 |
| 8 | 2.281 | 0.281 | 4.25 | 40.00 | 3.06 | 2.17 | 0.71 | 0.60 | 8.05 |
| 8 | 2.000 | 0.203 | 2.96 | 28.23 | 3.09 | 1.06 | 0.60 | 0.50 | 5.40 |
| 7 | 2.297 | 0.297 | 4.10 | 29.51 | 2.68 | 1.71 | 0.65 | 0.65 | 7.30 |
| 7 | 1.969 | 0.187 | 2.64 | 18.46 | 2.64 | 0.90 | 0.58 | 0.48 | 4.90 |
| 6 | 2.25 | 0.25 | 3.29 | 18.37 | 2.36 | 1.46 | 0.67 | 0.66 | 5.44 |
| 6 | 1.75 | 0.217 | 2.27 | 11.67 | 2.27 | 0.59 | 0.51 | 0.46 | 3.96 |
| 5 | 2.00 | 0.25 | 2.73 | 10.29 | 1.93 | 0.86 | 0.56 | 0.61 | 4.60 |
| 5 | 1.625 | 0.218 | 1.88 | 6.67 | 1.88 | 0.37 | 0.45 | 0.42 | 3.29 |
| 4 | 1.719 | 0.25 | 2.15 | 5.16 | 1.55 | 0.54 | 0.50 | 0.53 | 3.15 |
| 4 | 1.562 | 0.218 | 1.75 | 4.14 | 1.54 | 0.41 | 0.48 | 0.45 | 2.37 |
| 3 | 1.531 | 0.218 | 1.52 | 2.03 | 1.16 | 0.32 | 0.46 | 0.51 | 1.89 |
| 2¼ | 1.375 | 0.25 | 1.13 | 0.80 | 0.85 | 0.21 | 0.43 | 0.46 | 1.13 |

these two directions. Let $x$ be this distance, then by the application of the principles of moment of inertia (see Text-book on Mechanics of Materials, Arts. 23 and 51) there results equation

$$2 \times 64.34 = 2[2.47 + 5.40(\tfrac{1}{2}x - 0.67)^2]$$

whose solution gives $x = 8\tfrac{1}{2}$ inches. If, however, the flanges are turned outward the distance between the backs for equal moments of inertia will be found to be $5\tfrac{1}{2}$ inches nearly.

As a second example of the use of the table let a chord section be composed of two heavy 9-inch channels placed back to back 6 inches apart with a plate $\tfrac{1}{2}$ inch thick and 12 inches wide riveted upon the top (see Plate XII for a similar case). The total section is then $2 \times 5.40 + 12 \times \tfrac{1}{2} =$ 16.80 square inches. The distance $c$ of the centre of gravity from the lower side of the channel flanges is found by $16.80c = 10.80 \times \tfrac{9}{2} + 6(9 + \tfrac{1}{4})$ or $c = 6.20$ inches. The moment of inertia about a horizontal axis through this centre of gravity is $2(64.34 + 5.40 \times 1.70^2) + \tfrac{1}{12} \times 12 \times (\tfrac{1}{2})^3 + 6 \times 3.05^2 = 215.83$. The moment of inertia about a vertical axis through the centre of gravity is $2(2.47 + 5.40 \times 3.67^2) + \tfrac{1}{12} \times \tfrac{1}{2} \times 12^3 = 222.40$, and hence the column tends to bend perpendicular to the horizontal axis. The square of the least radius of gyration is $r^2 = 215.83 \div 16.80 = 12.85$ square inches.

## ART. 18. EYEBARS.

The proportions of eyebar heads have been determined mostly by experiment and experience, the theoretical discussions being quite difficult and uncertain on account of the complex stresses. In the following table are given the proportions adopted by the Pencoyd Bridge and Construction Company as their standards for steel eyebars, which are hydraulic forged. The dimensions adopted are such that all parts of the bar will be very closely of the same strength when tested to destruction. The maximum sizes of pin holes given in the table allow an excess in sectional area of the head on the line *SS* (in Fig. 14) over that of the body of the bar of 33 per cent for diameters of pins not larger than the width of the bar, and 36 per cent for pins of larger diameter. In these bars the thickness of the eye is the same as that of the body of the bar.

When bars are thicker than the dimensions given, $1\frac{1}{2}$ inches extra should be added to the additional length $L$ for each eye, but the thickness should not exceed $1\frac{3}{8}$ inches for the 7- and 8-inch bars, and one inch for the others.

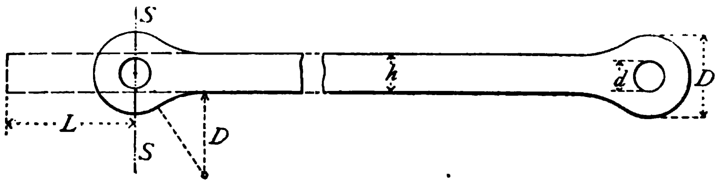



FIG. 14.

| $h$ Width of Bar. | $t$ Minimum Thickness of Bar. | $D$ Diameter of Head. | $d$ Diameter of largest Pin Hole. | $L$ Additional Length of Bar beyond Centre required to form one Head. |
|---|---|---|---|---|
| 3″ | $\frac{3}{4}$″ | 7″ | 3″ | 15″ |
| 3 | $\frac{3}{4}$ | 8 | $3\frac{7}{8}$ | $17\frac{1}{2}$ |
| 4 | $\frac{3}{4}$ | $9\frac{1}{2}$ | $4\frac{1}{8}$ | 18 |
| 4 | $\frac{3}{4}$ | $10\frac{1}{2}$ | $5\frac{1}{16}$ | $20\frac{1}{2}$ |
| 5 | $\frac{3}{4}$ | $11\frac{1}{2}$ | $4\frac{13}{16}$ | $19\frac{1}{2}$ |
| 5 | $\frac{3}{4}$ | $12\frac{1}{2}$ | $5\frac{11}{16}$ | $23\frac{1}{4}$ |
| 5 | $\frac{3}{4}$ | 13 | $6\frac{3}{16}$ | $26\frac{1}{2}$ |
| 6 | $\frac{7}{8}$ | $13\frac{1}{2}$ | $5\frac{1}{2}$ | 23 |
| 6 | $\frac{7}{8}$ | $14\frac{1}{2}$ | $6\frac{5}{16}$ | $26\frac{1}{4}$ |
| 7 | $\frac{15}{16}$ | 16 | $6\frac{11}{16}$ | $26\frac{3}{4}$ |
| 7 | $\frac{15}{16}$ | 17 | $7\frac{1}{2}$ | $30\frac{1}{4}$ |
| 7 | $\frac{15}{16}$ | 18 | $8\frac{1}{2}$ | $36\frac{3}{4}$ |
| 8 | 1 | 17 | $6\frac{5}{8}$ | $26\frac{3}{4}$ |
| 8 | 1 | 18 | $7\frac{3}{8}$ | 30 |
| 8 | 1 | $18\frac{1}{2}$ | $7\frac{7}{8}$ | $33\frac{3}{4}$ |

The flexural stress in an eyebar due to its own weight,* though often considerable, is less than that in a beam simply supported, since the tension reduces the bending moment. If $\Delta$ be the actual deflection of the bar when under a tension $P$, and $W$ be its total weight and $l$ its length, the bending moment at the middle is $M = \frac{1}{8}Wl - P\Delta$. Now let $t$ be the thickness and $h$ the width of the bar, $S$ the maximum horizontal tensile unit stress due to flexure, and $E$ the coefficient of elasticity of the material. Inserting the values of $M$ and $\Delta$ (from Arts. 21 and 37, Mechanics of Materials), the expression for this flexural unit stress is found to be

$$S = \frac{3WEhl}{4Eth^3 + 5Pl^2},$$

and the total maximum unit stress then is $\frac{P}{th} + S$.

For example, let a steel eyebar be 18 feet or 216 inches long, 1 inch thick, 8 inches wide, and be under a tension of 80 000 pounds, $E$ being 29 000 000 pounds per square inch. The weight of the bar is $8 \times 10 \times 6 \times 1.02 = 490$ pounds. Then all quantities inserted in the formula give $S = 944$ pounds per square inch, and the total maximum unit stress is 10 944 pounds per square inch. In designing very long eyebars the effect of the flexural stress due to their own weight cannot be neglected.

## ART. 19. STANDARD BRIDGE FLOOR, PA. R.R.

The track consists of the rails, cross-ties, spikes, guard-rails, bolts, washers and splices. Its weight, as a part of the dead load of a bridge, is determined in accordance with the specifications adopted in each case. Some specifications give the aggregate weight of a single track to be taken as 400 or 450 pounds per linear foot, while others allow 60 pounds per linear foot for the weight of rails, spikes and joints, or 100 pounds per linear foot for the weight of rails, iron guard-rails,

* See Chap. X, Modern Framed Structures, by Johnson, Bryan and Turneaure.

spikes and joints, the weight of the remaining parts being computed. Sometimes the sizes of the cross-ties are given for different spacings of the stringers, while in other cases the cross-ties are to be designed for given unit stresses in the outer fiber.

The standard specification of the Pennsylvania Railroad for wrought-iron bridges directs that a uniform load of 165 pounds per linear foot of track is to be assumed to cover the weight of rails, guard-rails, splices, bolts, etc., the weight of the cross-ties being computed separately. It also contains the

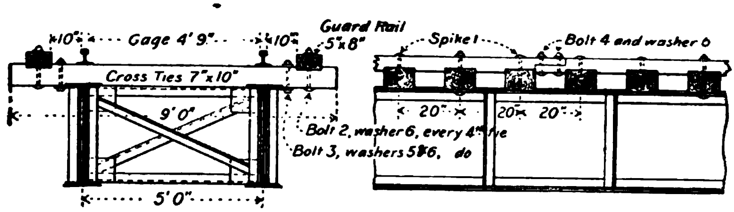



FIG. 15.

| Clear Span. | Length of Girder over all. | Railroad Spikes required. | Cross-ties required. | Length of Guard-rail. | Cross-ties and Guard-rails. |
|---|---|---|---|---|---|
| Feet. | Feet. | Number. | Number. | Feet. | Feet B.M. |
| 20 | 23 | 68 | 17 | 55 | 1 076 |
| 25 | 28 | 80 | 20 | 65 | 1 267 |
| 30 | 33 | 92 | 23 | 75 | 1 458 |
| 35 | 38 | 104 | 26 | 88 | 1 658 |
| 40 | 44 | 116 | 29 | 100 | 1 856 |
| 45 | 49 | 132 | 33 | 110 | 2 099 |
| 50 | 54.5 | 144 | 36 | 124 | 2 303 |
| 55 | 59.5 | 156 | 39 | 134 | 2 494 |
| 60 | 64.67 | 168 | 42 | 141 | 2 675 |
| 65 | 70 | 180 | 45 | 155 | 2 879 |
| 70 | 75 | 192 | 48 | 165 | 3 070 |
| 75 | 80 | 204 | 51 | 177 | 3 268 |
| 80 | 86 | 220 | 55 | 189 | 3 518 |
| 90 | 96 | 244 | 61 | 212 | 3 909 |
| 100 | 106 | 268 | 67 | 232 | 4 291 |

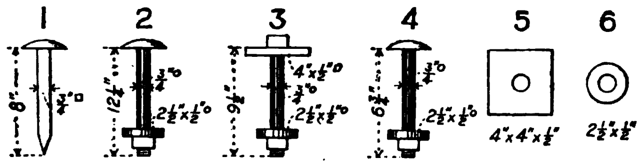



FIG. 16.

| Clear Span. | Spike 1. | | Bolt 2. | | Bolt 3. | | Bolt 4. | | Washer 5. | Washer 6. |
|---|---|---|---|---|---|---|---|---|---|---|
| Feet. | No. | Wt. | No. | Wt. | No. | Wt. | No. | Wt. | No. | No. |
| 20 | 24 | 32 | 10 | 25 | 10 | 57.5 | 4 | 7 | 10 | 24 |
| 25 | 28 | 37 | 12 | 30 | 12 | 69 | 4 | 7 | 12 | 28 |
| 30 | 34 | 45 | 12 | 30 | 12 | 69 | 4 | 7 | 12 | 28 |
| 35 | 38 | 50.5 | 14 | 35 | 14 | 80.5 | 8 | 14 | 14 | 36 |
| 40 | 42 | 55.75 | 16 | 40 | 16 | 92 | 8 | 14 | 16 | 40 |
| 45 | 48 | 63.5 | 18 | 45 | 18 | 103.5 | 8 | 14 | 18 | 44 |
| 50 | 54 | 71.5 | 20 | 50 | 20 | 115 | 12 | 21 | 20 | 52 |
| 55 | 58 | 77 | 20 | 50 | 20 | 115 | 12 | 21 | 20 | 52 |
| 60 | 62 | 82 | 22 | 55 | 22 | 126.5 | 12 | 21 | 22 | 56 |
| 65 | 66 | 87.5 | 24 | 60 | 24 | 138 | 12 | 21 | 24 | 60 |
| 70 | 70 | 93 | 26 | 65 | 26 | 149.5 | 12 | 21 | 26 | 64 |
| 75 | 74 | 98 | 28 | 70 | 28 | 161 | 16 | 28 | 28 | 72 |
| 80 | 80 | 106 | 30 | 75 | 30 | 172.5 | 16 | 28 | 30 | 76 |
| 90 | 90 | 119 | 32 | 80 | 32 | 184 | 20 | 35 | 32 | 84 |
| 100 | 100 | 132 | 36 | 90 | 36 | 207 | 20 | 35 | 36 | 92 |

All weights in this table are expressed in pounds.

following references to details which are shown in Figs. 15 and 16:

"Cross-ties to be of white oak, having a width of ten inches and a minimum depth of seven inches, and spaced not over twenty inches apart between centers, with every fourth tie bolted down by three-quarter-inch bolts having round flat heads, and two hexagon nuts each at upper end. When track is curved the outer rail to be elevated as may be required. In the case of deck bridges with wooden floor beams, when the distance between centers of supports exceeds six feet, the floor beams (ties) are to be made proportionately heavier.

"Guard-rails of long-leaved southern pine or white pine, five by eight inches, are to be placed ten inches in the clear outside of each track-rail; to be notched one and one half inches over the cross-ties, secured to every fourth tie by a three-quarter-inch bolt having flat round head, two hexagon nuts at upper end and flat wrought washer, and to all other ties by three-quarter-inch square wrought spikes. Splices to guard-rails to be twelve inches long placed between ties with the joint horizontal, two three quarter-inch bolts with flat round heads, wrought washers, and two hexagon nuts at upper end being used for each."

The preceding tables give the quantities of material required for a single track of this standard floor for plate-girder bridges. The number of spikes for the rails is four times the number of cross-ties. If $l$ is the length of the girder over all, and $c$ the distance center to center of cross-ties, the number of cross-ties equals $\frac{l}{c} + 3$. The number of bolts 2 and 3 (see Fig. 15) is $2\left(\frac{l}{4c} + 1\right)$. The number of bolts 4 depends on the number of guard splices. The number of spikes 1 equals

$$4\left[\left(\frac{l}{c} + 3\right) - \left(\frac{l}{4c} + 1\right)\right].$$

Taking the weight of the white-oak cross ties as $4\frac{1}{4}$ pounds per foot board measure, and that of the long-leaved southern pine guard rails as $3\frac{3}{4}$ pounds, the actual weight of 100 feet of single track when comparatively light rails are used may be computed as follows:

| | |
|---|---|
| 60 cross ties, 3150 feet B.M., @ $4\frac{1}{4}$ pounds... | 13 388 pounds. |
| 2 guard rails (666.7 − 100) feet B.M., @ $3\frac{3}{4}$ pounds.............................. | 2 125 |
| 90 spikes (1), 30 bolts (2), 30 bolts (3), 20 bolts (4), 30 washers (5), and 80 washers (6).... | 402 |
| 2 rails @ 60 pounds per yard.............. | 4 000 |
| 240 spikes @ $\frac{1}{2}$ pound...................... | 120 |
| 6 rail splices @ $13\frac{1}{2}$ pounds................ | 81 |
| Total............................ | 20 116 pounds. |

This gives an average weight of 201 pounds per linear foot, while if the weight exclusive of cross ties be taken at 165 pounds per linear foot the weight of track will be $134 + 165 = 299$ pounds per linear foot. The heaviest standard track contains rails weighing 100 pounds per yard.

## ART. 20. EXPANSION BEARINGS.

For bridges whose span does not exceed 75 feet, it is customary to provide for changes of length due to temperature by allowing the bearing plate at one end to slide on the bed plate. For larger spans an expansion bearing is provided, which generally consists of a nest of rollers of small diameter similar to that shown on Plate III. The diameter $d$ of the rollers is usually determined by the allowable pressure in pounds per linear inch and which in practice is taken equal to $K\sqrt{d}$, the value of $K$ (given in the leading specifications) varying from 500 to 750 when either or both the bearing plate and rollers are wrought iron, and 750 to 1000 when both are made of steel. As usually constructed and maintained it is by no means infrequent to find such bearings inefficient owing to the accumulation of dirt and rust.

In pin-connected trusses the stress is transferred through the pin to the pedestal and by it to the friction rollers (see Plate IV). The use of a pedestal in connection with a plate girder of long span is shown in Fig. 32 and is briefly described in Art. 42.

On Plate I is shown a standard expansion bearing, designed by GEORGE S. MORISON, which contains some valuable improvements over the forms containing segmental rollers which are used in Europe. When rollers of large diameter are employed, those parts which are not needed are removed, thus allowing their centers to be spaced much closer. To prevent the rollers from tipping over, the side plates which hold them in position by means of bolts are provided with hooks at each end, which, however, allow a linear movement $y =$ span $\div$ 3000 in both directions from the mean position. The steel rollers, which are 12 inches high, rest on a "rail plate" consisting of T-rails riveted to a plate with their tops afterwards planed and polished. The rail plate is bolted to the cast base which is directly supported by the pier masonry. When the dust accumulates it may be readily removed by passing a long-handled brush between the rails.

On top of the rollers rests a steel casting with its lower surface polished, and this in turn supports another casting by means of a block of polished steel called a rocker plate. The rocker plate fits into a socket in each casting, the surfaces of contact being segments of horizontal circular cylinders whose axes are respectively parallel and perpendicular to the direction of the rollers. The radius of curvature in each case equals the length of one side of the square rocker plate. The upper casting sustains the pedestal which is connected with the end pin of the truss.

The object of the rocker plate is to allow the bridge to adjust itself when erected so that the bearing on the rollers, and hence also that on the bed plate, may be uniform. This eliminates the unequal distribution of load which would otherwise be caused by imperfect workmanship in the construction of the truss and its supports.

The variable dimensions of the details, the linear bearing, and the safe load for bearings containing from 3 to 12 rollers, are given in the table on the opposite page. The form and dimensions of other details are shown on the drawing on Plate I. The half side elevation on the upper right-hand corner of the plate is that of the pedestal at the fixed end of the span.

In order that the end posts may have the necessary trans-

| Number of Rollers. | Number of Rails. | *a* Length of Rollers. | *b* Side of Rocker Plate. | *c* $\frac{a-(b+1.5)}{4}$ | Total Linear Bearing. | Safe Load at 3000 pounds per Linear Inch. |
|---|---|---|---|---|---|---|
| | | Inches. | Inches. | Inches. | Inches. | Pounds. |
| 3 | 6 | 17.5 | 4 | 3.00 | 45 | 135 000 |
| 4 | 8 | 23.5 | 5 | 4.25 | 80 | 240 000 |
| 5 | 10 | 29.5 | 6 | 5.50 | 125 | 375 000 |
| 6 | 12 | 35.5 | 8 | 6.50 | 180 | 540 000 |
| 7 | 14 | 41.5 | 9 | 7.75 | 245 | 735 000 |
| 8 | 16 | 47.5 | 10 | 9.00 | 320 | 960 000 |
| 9 | 18 | 53.5 | 11 | 10.25 | 405 | 1 215 000 |
| 10 | 20 | 59.5 | 12 | 11.50 | 500 | 1 500 000 |
| 11 | 22 | 65.5 | 13 | 12.75 | 605 | 1 815 000 |
| 12 | 24 | 71.5 | 14 | 14.00 | 720 | 2 160 000 |

verse stiffness they are preferably connected by an end floor beam which is riveted to them after the bridge is swung.

Fig. 17 shows a joint designed by Mr. MORISON for a viaduct. The alternate girders project a short distance beyond the posts

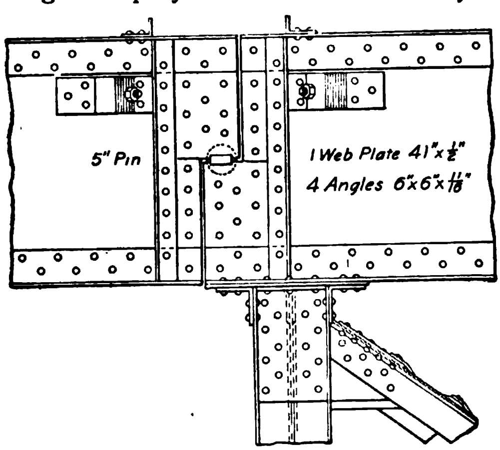



FIG. 17.

and the intermediate girders are supported on these projecting cantilevers. The object of the bearing is to provide not only for the longitudinal expansion and contraction, but also for deflection. It consists of a split pin, or two half-pins, and a key located in the neutral surface of the girders. The details are shown in Fig. 18, the dimensions given being those of the

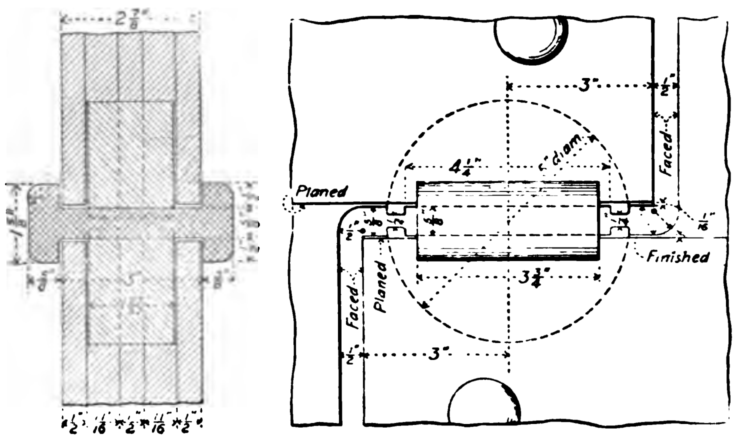



FIG. 18.

bearings in the Approach Viaduct of the Bellefontaine Bridge on the St. Louis, Keokuk and Northwestern Railroad.

The steel pin has a diameter of 5 inches and a length of $1\frac{27}{32}$ inches. In each girder it bears against the web and two reinforcing plates, and is held in position by two additional plates, one on each side of and extending over one half the depth of the girder. The clearance is $\frac{1}{32}$ inch. The pin is split and slotted to receive a phosphor-bronze key, the longitudinal movement allowed being $\frac{1}{2}$ inch. The projecting ends of the key keep it in place. All surfaces of the half-pins and key are finished.

For other forms of expansion bearings for viaducts, see a paper by J. W. SCHAUB in Transactions American Society of Civil Engineers, 1893, Vol. XXVIII, pp. 309–322, and an article on the Quaker City and Northeastern Elevated Railways, Philadelphia, Pa., in Engineering News, May 25, 1893, Vol. XXIX, p. 478. An application of this standard bearing is given in an article entitled Special Structural Details of the Memphis Bridge, in Engineering News, Sept. 7, 1893, Vol. XXX, p. 196.

## ART. 21. CONVENTIONAL SIGNS ON DRAWINGS.

Full lines show that they are visible, while invisible lines are represented by a series of dashes of equal length. In order to distinguish between invisible lines of the structure or object and the projecting lines it is desirable to use dashes about $\frac{1}{8}$ inch long for the former and about one third as long for the latter. The appearance of a drawing is materially improved by making the spaces between the dashes uniform. In general these spaces should measure about $\frac{1}{32}$ inch or a little less than the smaller dashes. If the spaces are longer than the dashes a line loses its apparent continuity if it is placed close to other lines of a similar character.

Feet are indicated by a prime (′) and inches by seconds (″), and these are usually placed on dimension lines having arrow points at the ends. These lines should be of two kinds: first, those marking the points, lines or sections between which the measurement is to be recorded; and second, those drawn at right angles to the preceding lines, with an arrow at each end and the dimension marked at the middle. The former should have the same form as projecting lines, while the latter should be distinguished from both projecting lines and invisible lines of the structure by using very short dashes or elongated dots and spaces nearly or quite $\frac{1}{8}$ inch long. In constructing these lines the pen should be opened about twice as far as for the ordinary lines constituting the greater part of the drawing.

Center lines of plans, elevations or sections, or lines marking the position of sections whose forms are shown elsewhere, are appropriately indicated by the usual convention for traces of auxiliary planes, consisting of very long dashes, say $\frac{3}{8}$ inch, with two dots between them. Center lines of members or rivet lines are indicated either by very light full lines in black or by red lines of ordinary weight. The red lines on tracing cloth usually give a faint line on the blue print which may be readily seen. The sizes of dashes and spaces given above are those suitable for bridge drawings whose scale ranges from $\frac{3}{4}$ to $1\frac{1}{2}$ inches to the foot, and should be modified accordingly for scales beyond these limits.

When drawings are to be shaded by making some of the lines heavier than others the following simple rule decides which lines are thus to be distinguished, plans being treated the same as if they were elevations: If a line separates two surfaces and there is an offset perpendicular to, and toward, the plane of projection in passing from the left-hand or upper surface to the right-hand or lower surface the line (marking the offset) should be shaded. If the offset is in the opposite direction, that is, if the former surface is nearer the plane of projection (or farther from the observer) than the latter, the line is not to be shaded. When the offset is not perpendicular, as in the case of a bevelled or chamfered edge, the form is usually indicated by the presence of diagonals or curves at the ends of the chamfer. If the line marks a rounded edge its weight should be increased but slightly. Shading adds very materially to the realistic effect of a drawing and enables workmen not trained to the use of drawings to interpret them more readily. On account of the extra labor involved, shading is frequently omitted on shop drawings.

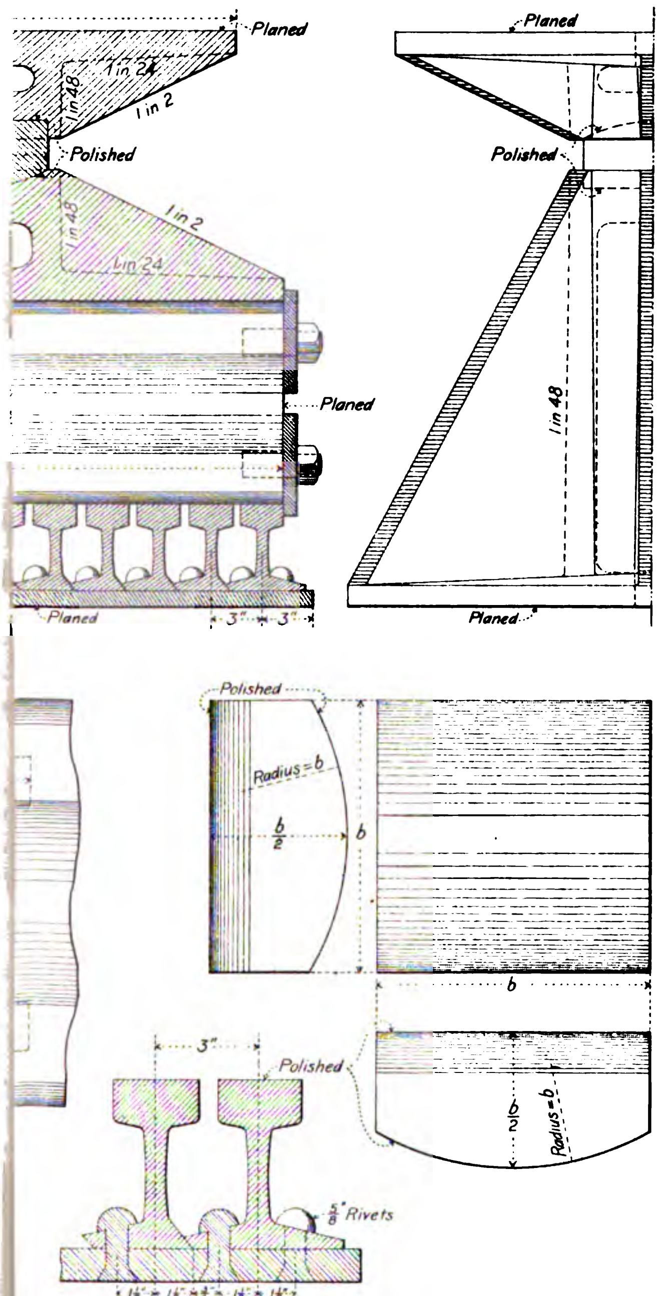
Clearness in detail drawings often demands that cylindrical, conical, spherical or other curved surfaces should be covered with shade lines spaced in accordance with the principles of shades and shadows in descriptive geometry.

Cross-sections are usually ruled with parallel lines drawn light and full as shown on Plate I (Art. 20). Sometimes a standard of section lining is adopted for different materials such as those shown in an article on this subject in Engineering News, Vol. XVIII, pp. 472–474, Dec. 31, 1887, but a preferable method for most purposes where it is necessary to make the distinction is to mark those parts composed of any material other than that which constitutes the bulk of the structure by placing the name of the material either on or adjacent to them. When the section is so small that ruling will not appear to be suitable the section is filled up solid. In order that adjacent sections so represented may appear distinct in form a small space is left between them although the shapes are really in perfect contact.

In order that a drawing may give proper directions for riveting it is necessary that some conventional signs shall be adopted for the rivets. A number of different systems of symbols have been devised, the two most prominent being respectively known as OSBORN'S code—which is published in OSBORN'S Tables of Moments of Inertia and in the Pocket Companion (Carnegie)—and the Pencoyd rivet signs published in the handbook of the Pencoyd Iron Works. Some other systems together with a discussion of existing and proposed standards may be found in the Engineering News, Vol. XXI, pp. 76, 119, 190, 191, 266, and 340, Jan.–Apr. 1889. OSBORN'S code has been adopted by a large number of bridge companies and engineers in the interest of uniformity of practice in the preparation of shop drawings. In the edition of 1893 of the Pocket Companion the lines indicating flattened heads are changed from the vertical to the inclined position. The attention of the student is particularly called to the different interpretations of the terms "inside" and "outside" with reference to this code in the communications published in the Engineering News as quoted above.

Where the sign of a rivet head is surrounded by a broken circle of larger diameter it represents the insertion of a washer to maintain uniform spacing between two angles acting together as a strut or tie. When the terms angles, channels, I beams, etc., are not marked on the drawing symbols are used having the form of the section. For further information regarding shop drawings see Art. 81.

ARING.



Planed
1 in 24
1 in 48
1 in 2
Polished
1 in 2
1 in 48
1 in 24
Planed
Planed
3"
3"
Planed
Polished
1 in 48
Planed
Polished
Radius = b
$\frac{b}{2}$
b
b
3"
Polished
$\frac{b}{2}$
Radius = b
$\frac{5}{8}$" Rivets

# CHAPTER IV.

## DESIGN OF A ROOF TRUSS.

### ART. 22. DATA AND SPECIFICATIONS.

The form of the truss is to be triangular, with vertical ties and inclined struts, the lower chord being horizontal throughout, and both ends of the truss fixed. The span of the truss is to be 50 feet, the distance between trusses 10 feet, and the rise of the peak 12½ feet, while the upper chord is to have eight panels. The roof covering is to consist of slate weighing 10 pounds per square foot, laid on sheathing 1 inch thick, supported by rafters spaced 2 feet center to center, these being carried by purlins placed at, or very near to, the panel points of the trusses. The vertical tie rods, washers, and shoe plates are to be of wrought iron, and the other parts of the trusses, as well as the purlins, rafters and sheathing, are to be of northern yellow pine whose weight is assumed to be 3 pounds per foot board measure. The snow load is to be taken at 25 pounds per square foot of horizontal projection, while the wind pressure is 40 pounds per vertical square foot.

The allowable unit stresses, expressed in pounds per square inch, shall be as follows: For the timber in tension only, 1300; in compression, 1100; in extreme fiber under flexure, 1200; for shear parallel to the fibre, 130; for compression on the side of the fibres, (*a*) when the entire side of a timber is compressed, 300; (*b*) when only half of the side is compressed, 400; (*c*) under a washer, 500. For the wrought iron in tension, 12 000; in extreme fiber under flexure, 12 000; for rivets in shear, 9000; in bearing, 18 000; and in the outer fiber of bolts under flexure, 15 000.

The lengths of the various truss members are now computed and checked graphically. The results are as follows, measurements being center to center: Upper chord, 27′ 11$\frac{7}{16}$″; each division of upper chord, 6′ 11$\frac{7}{8}$″; diagonals, 6′ 11$\frac{7}{8}$″, 8′ 10$\frac{1}{16}$″, and 11′ 3$\frac{1}{4}$″; verticals, 6′ 3″, 9′ 4$\frac{1}{2}$″, and 12′ 6″. The angle of inclination $\alpha$ is 26° 34′, sec $\alpha = 1.118$, cos $\alpha = 0.894$, and sin $\alpha = 0.447$.

## ART. 23. RAFTERS AND PURLINS.

If the rafters are spaced far apart the thickness of the sheathing must also be computed for safety under flexure, but in this case no such investigation is necessary, as it is evident that one-inch sheathing with a span of two feet has a considerable excess of strength.

The span of one division of a rafter is the distance between the purlins or 6.99 feet. The area supported by it is $2 \times 6.99 = 13.98$ square feet. The vertical load is the weight of snow, slate and sheathing covering this area in addition to the weight of the rafter. Excluding the last item the weight is $(25 \cos \alpha + 10 + 3)13.98 = 494$ pounds. Adding about 1.5 pounds per square foot for the assumed weight of rafter, the component normal to the rafter is $(494 + 21) \cos \alpha = 460$ pounds. The normal wind pressure is 23.8 pounds per square foot (see Part I, Art. 13, or Part II, Art. 19), and the wind load on this area is $23.8 \times 13.98 = 333$ pounds. The bending moment in the rafter is $\frac{1}{8}(460 + 333)6.99 \times 12 = 8320$ pound-inches. As rafters are usually made 2 inches wide the resisting moment is $1200 \times 2 \times d^2 \div 6 = 400d^2$, where $d$ is the depth in inches. Equating the moments and solving, $d$ is found equal to 4.6 inches. The usual merchantable sizes are $2 \times 4$, $2 \times 6$, $2 \times 8$ inches, etc., but $2 \times 5$ inches can now also be obtained in some places. The section of $2 \times 6$ inches will be selected. The compression due to the longitudinal component of the vertical loads, which amounts to 124 pounds at the middle, was neglected above, but is covered by the increase of depth. Each division of the rafter was moreover treated as a simple beam, while it is generally continuous over two or three divisions.

The purlin is a beam subject to concentrated loads 2 feet apart whose magnitude is the resultant of the vertical load of 515 pounds and the normal load of 333 pounds, which is found to be 826 pounds. Let the rafters be so placed that one will come directly over each truss, then the bending moment in the purlin is 59 460 pound-inches due to the concentrated loads alone. The resisting moment of a timber $5 \times 8$ inches is $1200 \times 5 \times 8^2 \div 6 = 64\,000$ pound-inches, and the bending moment due to its own weight of 100 pounds is 1500 pound-inches. This size therefore answers the condition. Strictly, the purlin should be so placed that its sides are parallel to the resultant referred to above, but in this case, where the purlin has an excess of strength, a slight modification will be made, allowing its sides to be perpendicular to the upper chord of the truss and thus avoiding the necessity of notching in order to give to the rafters an even bearing on the purlin. When the roof is steep, however, this modification should not be allowed.

The purlins are held in position and in turn hold the trusses laterally by double notching on the upper chord, the notch on each timber being about an inch deep. A block is also spiked to its lower side to resist its tendency to overturn under the influence of the longitudinal force in the rafters.

For descriptions of some celebrated timber roofs see American Architect, June 6 and 13, 1885. For an example of the purlins on an iron roof see Chap. X.

## ART. 24. LOADS AND STRESSES.

The weight of the truss, according to the formula in Part II, Art. 16, is 1500 pounds, or 187 pounds per apex. The area of roof covering supported by one apex of the truss is $6.99 \times 10 = 69.9$ square feet and the corresponding weight of the slate and sheathing is $13 \times 69.9 = 909$ pounds. The weight of five rafters $2 \times 6$ inches and 6.99 feet long is 105 pounds, and of one purlin $5 \times 8$ inches and 10 feet long is 100 pounds. The dead apex load for the truss is therefore 1300 pounds. The snow apex load is $6.25 \times 10 \times 25 = 1562$ pounds, and the wind apex load $6.99 \times 10 \times 23.8 = 1664$ pounds. The maximum stresses in the members of the truss are now computed by either the analytic method of Part I or the graphic method of Part II, and the results are marked on the skeleton diagram in Plate II.

## ART. 25. MAIN SECTIONS.

It is proposed to use rods for the vertical ties without upset ends, and hence their areas at the root of the thread must equal or slightly exceed those obtained by dividing their stresses by the allowed unit stress of 12 000 pounds. The net areas are 0.197, 0.394, and 0.948 square inches, and the corresponding diameters are found, by referring to any of the pocketbooks, to be $\frac{5}{8}$, $\frac{7}{8}$, and $1\frac{3}{8}$ inches respectively.

As the structural shapes for designs in this text-book will be selected from the "Pocket Companion containing useful information and tables appertaining to the use of steel as manufactured by the Carnegie Steel Company, Limited, Pittsburgh, Pa., 1893," the reference to its tables will now be made. From the table in the Pocket Companion beginning on page 225, the diameters at the root of the thread are 0.501, 0.709, and 1.10 inches, and the table on page 215 gives the corresponding diameters of the bolts. A rod $\frac{5}{8}$ inch in diameter will also be placed below the apex next to the support in order to sustain the lower chord. Its section would not need to be so large, but it is taken of the same size as the next one in order not to increase the number of sizes to be manufactured. The rods will have nuts at both ends to facilitate erection, whose sizes are of the usual standards as given in the handbooks (see Pocket Companion, page 211).

The net area of the lower chord is $28\,370 \div 1300 = 21.8$ square inches, but its gross area must be larger as notches are to be cut into its upper surface near the end to receive the shoe plates, and bolts must be passed through it near the notches. Although a smaller net area is required near the middle of the truss, its section will be made uniform and the splices in the chord located in that vicinity. It is furthermore desirable in small trusses like this one to have the sides of all its wooden members flush with each other, and as this uniform width of the timbers will be the longer dimension of the section of the struts it must be some even inch as 4, 6, or 8. The market sizes of timbers have the smaller dimension in either an even or an odd number of inches, while the larger one is, as a rule, an even number of inches only. The dimensions referred to are the nominal ones, the actual being usually a little scant. The best width will therefore usually be such that the upper chord is a little deeper than its width and the lower chord of the same depth as its width or a little less. This consideration will make the width six inches. As the net section of the lower chord is nearly $6 \times 4$ inches the gross section must either be 5 or 6 inches deep. This will be determined when the shoe is designed.

RANKINE'S column formula is to be used for the compression members, the allowable unit stress for flat ends being

$\dfrac{1100}{1+\dfrac{1}{3000}\cdot\dfrac{l^2}{r^2}}$, in which $l$ is the length in inches and $r$ is the least radius of gyration in inches. For a division of the upper chord $l = 83.9$ inches, and for a width of 6 inches, $r^2 = 6^2 \div 12 = 3.0$, while the unit stress is 617 pounds. The area is therefore $29\,860 \div 617 = 48.4$ square inches, or $6 \times 8$ inches. This section is just a trifle scant, but as the chord is fixed at the shoe in such a manner as practically to reduce the length between centres as taken above by about one foot it will answer. A timber $6 \times 6$ inches would be strong enough for the third and fourth panels, but the chord will be made uniform throughout for the sake of its appearance.

The ends of the struts are regarded as intermediate in condition between flat and round ends so that the fraction $\frac{1}{3000}$ in RANKINE'S formula for flat ends is replaced by $\frac{1.78}{3000}$. Assuming the least dimension of the longest strut to be 6 inches, the unit stress is found to be 239 pounds and the required area $8500 \div 239 = 35.6$ square inches, making the section $6 \times 6$ inches. Assuming five inches for the least width of the next strut the area required is 25.4 square inches, which takes a timber $5 \times 6$ inches. The shortest strut must be $4 \times 6$ inches in section. In any of these cases if the required area would allow a smaller dimension than the least one assumed a revision would be necessary using the new value of $r$.

## ART. 26. BEARING AREAS.

In order to resist the longitudinal components of the stresses in the struts with respect to the upper and lower chords, indents will be cut in them to receive the ends of the struts. As the pressure then acts against the fibers of the struts in a direction intermediate between endwise and sidewise the allowable unit stresses should vary correspondingly.

Passing from left to right the stresses for the lower ends of the struts may be taken at 1000, 900 and 800, and for their upper ends 800, 550 and 300 pounds. The components parallel to the upper chord of the stresses in the struts are 3190, 2130 and 1060 pounds respectively, requiring bearing areas of 3.5 to 4 square inches, which will be furnished by an indent $\frac{3}{4}$ inch deep. The horizontal components of the stresses in the first and second struts are equal and amount to 4730 pounds and therefore require indents $\frac{7}{8}$ inch deep. The greatest difference of stress in the middle struts is the magnitude of the wind stress, which equals 3350 pounds, its horizontal component being 1850 pounds. As this is resisted by the fibers on the farther side of the adjacent strut it should have an area of $1850 \div 300 = 6.2$ square inches, thus making the indent about one inch deep. If, later, the splice in the lower chord will be placed at the centre, it may be found desirable to change this arrangement.

To keep the struts from lateral displacement tenons about $1\frac{1}{4}$ inches in thickness and extending into mortices in the chords about $2\frac{1}{2}$ inches deep may be employed, or instead of that dowels $\frac{1}{2}$ or $\frac{3}{4}$ inch in diameter and about 6 inches long. The former plan will be adopted.

The bearing of the ends of the struts against the sides of the fibers in the chords also needs investigation, and if the bearing area of the longest strut, which is almost perpendicular to the upper chord, is sufficient the others will also be safe. The stress in this strut is 8500, calling for a bearing area of $8500 \div 300 = 28.3$ square inches. If the tenon is one inch thick the area of the shoulders is $(6 - 1\frac{1}{4})6 = 28.5$ square inches, which is sufficient.

The area required under the washers of the middle rod is $11\,370 \div 500 = 22.7$ square inches, which calls for a washer about $5\frac{5}{8}$ inches in diameter (see Pocket Companion, pages

203–208). As this almost covers the entire width of timber the unit stress will be reduced to 400 pounds, and the bearing area now must be 11 370 ÷ 400 = 28.4 square inches. Adding the area of the hole in the washer, which is $1\frac{7}{16}$ inches in diameter, the total area is 28.4 + 1.6 = 30.0 square inches. Using a rectangular plate of the same width as the chord its length must be 5 inches. At the upper end of the rod, however, the plate not only serves to distribute the pressure from the rod but also to hold the extremities of the upper chords in position, hence its length will be increased to 7 inches and bent so as to provide a horizontal bearing for the nut. If the plate be regarded as a beam 6 inches wide and 7 inches long with the load concentrated in the middle and the reactions uniformly distributed on each side of the middle, a thickness of $\frac{7}{8}$ inch would be needed for a unit stress of 12 000 pounds, but as the reaction is really distributed on all sides of the rod and not simply on two sides, the thickness may be reduced somewhat, say to $\frac{3}{4}$ inch.

The bearing area under the $\frac{7}{8}$-inch rod is 4720 ÷ 500 = 9.4 square inches. The area of the $\frac{15}{16}$-inch hole in the washer is 0.7, making the total 10.3 square inches. Either a round washer $3\frac{5}{8}$ inches in diameter or a square one $3\frac{1}{4}$ inches wide will supply this gross area. The square form will be used as it allows more bearing area on its sides for the purlin on the chord. Tables in some of the handbooks for engineers state that wrought-iron washers should have a thickness of about one eighth of their diameter for the round, and about one seventh of their side for the square. It is not stated whether these ratios depend upon experimental results. This would make the washer $\frac{7}{16}$ inch thick. The washer for the $\frac{5}{8}$-inch rod is found to be $2\frac{1}{4}$ inches square and $\frac{5}{16}$ inch thick.

The normal component of the maximum pressure of the purlin on the chord is $(793 \times 5) + 100 \cos \alpha = 4055$ pounds, requiring a bearing area of 13.5 square inches. On the two sides of the larger washer, $3\frac{1}{4}$ inches square, is an area aggregating $(6 - 3.25)5 = 13.75$ square inches, and it is therefore sufficient.

## ART. 27. DESIGN OF END JOINT.

The foot of the upper chord will be held in position by a shoe consisting of two bent wrought-iron plates, each one having a tooth which shall engage in a notch in the lower chord timber. Let these plates be assumed to be $\frac{3}{4}$ inch in thickness, then the resisting moment of one plate is $12\,000 \times 6 \times 0.75 \times 0.75 \div 6 = 6750$ pound-inches. The pressure of the wood against one of the teeth whose depth is $d$ is $6 \times d \times 1100$ and causes a bending moment in it of $(6 \times d \times 1100 \times d \div 2)$ pound-inches. Equating these moments and solving, $d = 1.44$ or $1\frac{7}{16}$ inches. The total stress which will be taken by the two teeth is $2 \times 6 \times 1.44 \times 1100 = 19\,000$ pounds. The clear distance between them must provide a horizontal shearing surface whose strength equals the compressive strength of the side of one notch. Its length must be $9500 \div (6 \times 130) = 12.2$ inches, but as a bolt will pass through this surface its length will be made $12\frac{1}{2}$ inches. The lower chord must be extended the same distance beyond the outer tooth of the shoe plate.

The horizontal component of the stress in the upper chord is $29\,860 \cos \alpha = 26\,690$ pounds, hence a balance of $26\,690 - 19\,000 = 7690$ pounds must be taken by some other means. Bolts will be used whose direction is perpendicular to the upper chord. The longitudinal stress in these bolts must therefore be $7690 \div \sin \alpha = 17\,210$ pounds, which requires two bolts each having an area of $8605 \div 12\,000 = 0.72$ square inches. A bolt $1\frac{1}{4}$ inches in diameter will have an area at the root of the thread of 0.891, the area of the next smaller size being a little too small. These bolts require a bolster under the truss in order to avoid cutting into the lower chord and weakening it. It will be convenient to make the bolster of the same section as the chord. As the washers will have an inclined bearing on the fibers a unit stress of about 600 pounds may be employed and the bearing area is $8605 \div 600 = 14.3$ square inches. A washer 4 inches square will answer the purpose. The position of the bolts is shown on the drawing.

The stress in the lower chord requires a net area of $28\,370 \div 1300 = 21.8$ square inches. If the lower chord is 6 inches deep the net section at the notch is $(6 - 1.44)(6 - 1.25) = 21.7$ square inches. But the bolster requires one or two keys in order to keep it in position. These keys must transmit a stress of $28\,370 - 19\,000 = 9370$ pounds. If they are made $1\frac{1}{2}$ inches deep and their fibers are parallel to those of the chord each one can take $6 \times \frac{3}{4} \times 1100 = 4950$ pounds. This stress requires a net area of fibers in tension $4950 \div (6 \times 1300) = 0.64$ inch high and extending across the timber, and a shearing surface on the left of the key $0.64 \times 1300 \div 130 = 6.4$ inches long. Now unless the inner key is placed farther than 6.4 inches to the right of the inner notch the net area of the fibers which could transmit tension past the notch and bolts would be only $(6 - 1.44 - 0.64 - 0.75)(6 - 1.25) = 15.05$ square inches. But the stress thus to be transmitted is $28\,370 - 4950 = 23\,420$ pounds and requires $23\,420 \div 1300 = 18.02$ square inches, or about 3 square inches more, and this will demand an additional shearing length of $3 \times 1300 \div (130 \times 6) = 5.0$ inches. The clear distance between the inner notch and key must therefore be about $11\frac{1}{2}$ inches, while the bolster must extend at least $6\frac{1}{2}$ inches beyond the key. The outer key may be placed anywhere beyond the bolts as the net area of 18 square inches is provided for. The width of the keys should be at least double their thickness, as is shown by experiment, in order to prevent crushing the fibers by rotating the key. Using either selected southern yellow pine or white oak for the keys their unit shearing strength along the fiber may be taken 50 per cent greater than that of the main timber. The length of the key is hence made $4\frac{1}{2}$ inches. The pressure on the ends of the key is applied on the upper half of one end and the lower half of the other, producing a moment of rotation of $4950 \times \frac{1}{2}(1.5) = 3713$ pound-inches. This moment causes diagonally opposite halves of the sides of the key to press against the main timbers which act as beams of short span in transferring this pressure to the bolts on both sides of the key. If the pressure on each side of the key varies uniformly from zero at the middle to its maximum value at the end the horizontal distance between the centres of pressure is two thirds of the length of the key, or 3 inches. This gives the total pressure on one side of $3713 \div 3 = 1238$ pounds. The size of bolt necessary to resist this stress is smaller than practical considerations allow. The same diameter will be used as for the smallest tie rod.

The vertical component of the reaction is $29\,860 \sin \alpha = 13\,350$ pounds, and the bearing area of the truss on the wall plate is $13\,350 \div 300 = 44.5$ square inches. If the wall plate is 8 inches wide it will furnish the requisite bearing. The horizontal component of the reaction is 1680 pounds and may be conveniently provided for by a one-inch notch over the wall plate, which is fastened to the wall at regular intervals by anchor bolts. The truss may be held in place either by a few spikes or by means of two small bent plates fastened to the bolster by a bolt and to the wall plate by spikes or lag screws.

The purlin near the shoe is conveniently placed with its sides vertical and made sufficiently deep to afford the necessary bearing for the rafters.

## ART. 28. CHORD SPLICES.

If timbers for the upper chord cannot be conveniently secured of the full length, the splice should be placed as near as possible to one of the apexes. The half lap joint has the advantage of simplicity and should be about twice as long as the depth of the timber. The parts are united by two bolts, as shown on the drawing.

The splice in the lower chord will preferably be located at the center where the stress is least, and thus require chord timbers a little less than 26 feet long. A plain fish-plate joint with bolts will be used. The net area for tension must be $18\,940 \div 1300 = 14.6$ square inches, the area of compression of the wooden fibers against the bolts is $18\,940 \div 1100 = 17.2$ square inches, and the area required for shearing along the fibres is $18\,940 \div 130 = 145.7$ square inches. Unless the bolts are very small each bolt tends to shear the timber in two surfaces a little nearer together than the planes tangent to its own surface. The bolts are subject to flexure, the pressure of the timber against the bolts being regarded as uniformly distributed. In order that the fish plates shall be as strong as the main timbers connected by them each of the former must be half as thick as the latter. Let an investigation be made to see whether the main timbers will be strong enough if reduced to a thickness of 3 inches, and the plates $1\frac{1}{2}$ inches, so that their combined thickness may equal that of the section adopted for the lower chord.

The curve of moments for the bolts is shown in Fig. 19. The triangle *beh* would be the moment diagram if the uniform loads were regarded as concentrated at their centers of gravity. The parabolas *ac*, *cg*, and *gi* are drawn tangent to the sides of the triangle at the points *a*, *c*, *g* and *i*, their axes being parallel to the direction of the forces. (see Part II, Art. 10.) The diagram also shows that the maximum moment is the same as it would be if the forces in each half of the main timber were concentrated at their center of gravity. The value of the maximum moment is therefore the product of the tension in one of the fish plates and the distance between the center of a fish plate and the quarter point of the main timber, and if the fish plate is one half as thick as the main timber this distance equals the thickness of the fish plate. In the present example the moment is $9470 \times 1.5 = 14\,205$ pound-inches. Now assume that four bolts may be needed in each half of the

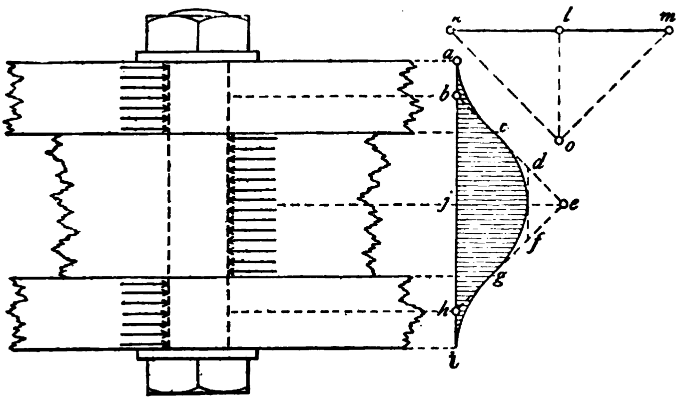



FIG. 19.

joint, then the resisting moment of one bolt is $14\,205 \div 4 = 3550$ pound-inches. The resisting moment of a bolt whose diameter is $d$ is $\dfrac{SI}{c} = \dfrac{S\pi d^3}{32}$, in which $S$ is the unit stress in the outer fiber, $I$ the moment of inertia of its cross-section, and $c$ the distance of the outer fiber from the neutral surface. The computed values for a large range of diameters are usually published in the manufacturers' handbooks. Referring to the Pocket Companion, on page 173 it is found that the bolt must be $1\frac{3}{8}$ inches in diameter for a unit stress of 15 000 pounds. This size bolt would leave a net area for tension in the main timber of $(6 - 1\frac{3}{8})3 = 13.9$ square inches. As this is only a little below the required area, let the thickness be increased to $3\frac{1}{4}$ inches, giving a net area of 15.0 inches, and making the combined thickness only $6\frac{1}{2}$ inches. This will increase the bending moment in a bolt to $9470 \times 1\frac{5}{8} \div 4 = 3850$ pound-inches, and as this is about equal to the resisting moment of a $1\frac{5}{8}$-inch bolt, this size will still answer. The compressive area is now $4 \times 1\frac{3}{8} \times 3\frac{1}{4} = 17.9$ square inches, which being just a trifle larger than that required shows that it is safe, and also that the number of bolts assumed is the minimum allowed. Of course a number of bolts larger than the minimum might be used, but it would not usually be economical in material. The shearing surfaces instead of extending to the surface of the bolt opposite the center are probably about one quarter of the diameter shorter. As there are eight shearing surfaces the length of each one is $145.7 \div (8 \times 3\frac{1}{4}) = 6\frac{1}{8}$ inches, making the distance of a bolt from the end of a timber $6\frac{1}{8} + (\frac{1}{4} \times 1\frac{3}{8}) = 6\frac{1}{2}$ inches, and the bolts $6\frac{1}{2} + (\frac{1}{2} \times 1\frac{3}{8}) = 6\frac{7}{8}$ inches center to center. On account of the rod at the middle of the truss the nearer bolt should be placed about 7 inches from the center. The length of the fish plates will then be $2(6\frac{1}{2} + 3 \times 6\frac{7}{8} + 7) = 5$ feet $8\frac{1}{4}$ inches. As these bolts are subject to very little tension, their heads and nuts would not need to have the standard thickness, and the smallest size washers may be employed, the nuts being drawn up simply to keep the parts in close contact. The holes should be bored of such a size that the bolts when driven in will fit tightly.

The fish plates will be given a projection at the middle to receive the shoulders of the adjacent struts and afford sufficient bearing, and the space between them receives the thick tenons of the struts. No mortise therefore is required to be cut in the chord.

Instead of using wooden fish plates and cutting down the main chord timber, it would be preferable to employ wrought-iron ones laid flush with the surface of the chord. This arrangement would require two bolts $1\frac{1}{16}$ inches in diameter in each half of the joint and spaced 7 inches center to center, the total bending moment in the bolts being in this case the same as before. The plates would be $\frac{1}{4}$ inch thick and about 2 feet 9 inches long.

Still another plan would use two plates $\frac{3}{8}$ inch thick and with a tooth at each end of the same dimensions as those used for the shoe. These plates would be $2 \times 12.2 = 24.4$ inches long between the teeth, or about 26 inches out to out after bending. If the parts are kept in position by two half-inch bolts in each half of the joint, there will still be enough net section for tension, but it will not allow the plates to be laid flush with the chord. This arrangement would be more expensive than the others and would require more careful workmanship to insure uniform bearing of the teeth.

## ART. 29. WEIGHT OF THE TRUSS.

The finished lengths of all truss members and details are marked on the drawing, and from these a complete bill of material may be readily prepared for any given number of trusses. Such market lengths of timber should be chosen for the various sections as may be cut up with the least waste.

With the aid of the tables in the Pocket Companion, beginning on pages 197, 203, 210 and 211, and remembering that the first two tables are for steel, which is two per cent heavier than wrought iron, the total net weight of material for one half of a truss is computed to be 1010 pounds, consisting of timber, 789 pounds; tie-rods, washers and nuts, 80 pounds; bolts and washers, 71 pounds; and bent plates, 70 pounds. If that part of the half truss whose weight belongs to the apex load at the support is deducted, the weight of timber is reduced to 684 pounds, and that of the bolts and washers to 40 pounds, while the tie-rods, etc., remain unchanged and the bent plates are entirely omitted. The total net weight for the $3\frac{1}{2}$ apexes is therefore 804 pounds, or 230 pounds per apex. This is 43 pounds more than was assumed, or a little more than 3 per cent of the entire dead panel load. No revision of stresses and sections is hence necessary.

The weight of one truss being 2020 pounds, and that of the rafters and roof covering for 10 feet in length (Art. 24) being 8900 pounds, the total weight of the roof for 500 square feet of floor area is 10 920 pounds, or nearly 2.2 pounds per square foot of horizontal area covered.

# CHAPTER V.

## DESIGN OF A PLATE-GIRDER BRIDGE.

### ART. 30. SPECIFICATIONS.

Let the design be that of a deck plate-girder bridge for a single-track railroad, the span being 80 feet between centres of bearing plates. The bridge is to be located on a straight track, and its material, with the exception of the track and the rivets, is to be medium steel. The specifications to be used are those of F. H. LEWIS contained in a pamphlet entitled Soft Steel in Bridges, first published in the Proceedings of the Engineers' Club of Philadelphia, Vol. IX, January, 1892. By permission, so much of the specifications as relates to the design of plate girders of the above span is here reprinted.

SPECIFICATIONS FOR FIRST-CLASS BRIDGE SUPERSTRUCTURE.

*General Description.*

2. Ties on tangents will be 8 inches by 10 inches, laid on 8-inch face and spaced 14 inches between centers; guard-rails will be 7 inches by 8 inches, spaced 8 feet between centers. . . . Ties will be notched $\frac{3}{4}$ inch over stringers, and guard-rails $\frac{3}{4}$ inch over ties. Guard-rails will be bolted to each end of every other tie; and ties and guard-rails will be secured to stringers by hook bolts at each end of every fourth tie.

3. For spans of 16 feet or less, rolled beams will be used, and from 16 feet to 100 feet, riveted plate girders. All spans over 100 feet will be pin-connected trusses.

4. Beams or deck girders on masonry will be spaced 7 feet 0 inches center to center (ties 10 feet long). . . .

10. All structures will be simple in design, and admit of accurate calculation of the stresses in each member.

18. Live loads will be as per diagram furnished by the Chief Engineer.

*Loading.*

19. The structure will be proportioned to carry the live loads as per diagram, and the live-load stresses will be the maximum stresses produced by the rolling load considered as stationary or as moving in either direction. . . .

20. The dead load shall consist of the entire structure, including the floor system and rails and fastenings. The weight of the ties, guard timbers, rails, spikes, etc., shall be taken at 400 pounds per linear foot for each track. The load of the structure when complete shall not exceed the dead load used in calculating the stresses.

*Wind in Trusses.*

22. The bottom lateral bracing in deck bridges and the top lateral bracing in through bridges must be proportioned to resist a uniformly distributed lateral force of 150 pounds per linear foot of bridge for all spans of 200 feet and under, and an additional force of 10 pounds per lineal foot for every 25 feet increase in length of span over 200 feet.

23. The bottom lateral bracing in through bridges and the top lateral bracing in deck bridges must be proportioned to resist a uniformly distributed force the same as above, and an additional force of 300 pounds per linear foot of bridge, which shall be treated as a moving load.

*Centrifugal Force.*

25. When the bridge is on a curve add to the maximum wind stresses a moving lateral stress equal to 3 per cent of the live load on all tracks (acting in the direction of centrifugal force) for each degree of curvature.

26. The effects of wind and centrifugal force in the lateral system of structures must be fully provided for at unit stresses given below.

*Longitudinal Bracing and Anchorage.*

27. Longitudinally the bracing of trestle towers and the attachments of the fixed ends of all trusses shall be capable of resisting the greatest tractive force of the engines or any force induced by suddenly stopping the assumed maximum trains, the coefficient of friction of the wheels upon the rails being assumed to be 0.20. . . .

*Temperature.*

28. Variations in length from change of temperature to the amount of 1 inch in 100 feet shall be provided for.

### *Calculations and Unit Stresses.*

29. All parts of the structure will be proportioned to sustain the maximum stresses produced by the live and dead loads specified above, and by the wind and centrifugal forces under special conditions provided in paragraphs 26, 32 and . . .

30. In calculating strains, conventional assumptions will be used throughout. The lengths of spans will be the distance between centers of end pins of trusses, and between centers of bearing plates of beams and girders. The length of stringers will be the distance between centers of floor beams, and the length of floor beams the distance between centers of trusses. The depth for calculation of girders will be the distance between centers of gravity of flange sections, provided it does not exceed the distance out to out of angles, in which case the latter amount shall be considered the depth.

### *Formulas for Unit Stresses.*

31. The following formulas for unit strains per square inch of net sectional area shall be used in determining the allowable working stress in each member of the structure.

*Tension Members.*

Medium Steel.

(*c*) Tension flanges of girders, net sections. See ¶ 82. $9000\left(1 + \frac{\text{min.}}{\text{max.}}\right)$.

*Compression Members.*

(*h*) Lateral struts * * * ........................ $12600 - 60\frac{l}{r}$.

In which formula, $l$ = length of compression member in inches, and $r$ = least radius of gyration of member in inches.

*Members subject to Alternate Tension and Compression.*

(*i*) For compression only.................. Use the formula above.

For the greatest stress............... $8400\left(1 - \frac{\text{max. lesser}}{2\text{ max. greater}}\right)$.

Use the one giving the greatest area of section.

(*j*) The compression flanges of beams and plate girders will have the same cross-section as the tension flanges.

*Shearing.*

| | Soft Steel. | Medium Steel. |
|---|---|---|
| (*l*) On pins and shop rivets............ | 6600 | 7200 |
| On field rivets...................... | 5200 | Will not be used. |
| In webs of girders.... ............. | 5000 | 6000 |

*Bearing.*

| | Soft Steel. | Medium Steel. |
|---|---|---|
| (*m*) On projected semi-intrados of main pin holes and rivet holes.... | 13 200 | 14 500 |

* * *

On bed plates of masonry, 250 pounds per square inch.

(*o*) *Coefficients of Friction* will be used as follows:

Wrought iron or steel on itself.................... 0.15
Wrought iron or steel on cast iron................. 0.20
Wrought iron or steel on masonry................. 0.25
Masonry on itself.................................. 0.50

32. In case the maximum stresses in chords, girder flanges, trestle posts, or the bending effects on posts due to wind or centrifugal force, shall exceed 25 per cent of stresses due to dead and live load, the sections will be increased until the total strain per square inch will not exceed by more than 25 per cent the maximum fixed for live and dead load only.

34. The effects of the weights of horizontal or inclined members in reducing their strength as columns must be provided for. It will also be considered in fixing the position of pin centers.

35. Plate girders shall be proportioned upon the supposition that the bending or chord strains are resisted entirely by the upper and lower flanges, and that the shearing or web strains are resisted entirely by the web plate.

36. The effective diameter of the driven rivet shall be considered the same as the diameter before driving. In deducting for rivet holes, the diameter of the hole will be considered $\frac{1}{8}$ inch greater than the rivet for full-headed rivets, and $\frac{1}{4}$ inch larger for countersunk rivets.

37. No compression member shall have a length exceeding 45 times its least width, and no post will be used in which $\frac{l}{r}$ exceeds 125.

## Details of Construction and Workmanship.

### *General.*

40. All details must be of approved forms and satisfactory to the Chief Engineer.

41. Preference will be had for such details as will be most accessible for inspection, cleaning, and painting.

42. No shape iron weighing less than 6 pounds per linear foot will

be used, nor any iron less than $\frac{3}{8}$ inch thick, nor any bar of less than one square inch section. No angle smaller than 3 inches by 3 inches will be used in girders or truss members, or in any member having $\frac{7}{8}$-inch rivets. No angle smaller than $2\frac{1}{2}$ inches by $2\frac{1}{2}$ inches will be used in any part of bridge structures. End angles carrying stringers and floor beams will be at least $\frac{1}{2}$ inch thick.

43. All bed plates will be at least $\frac{3}{4}$ inch thick.

46. Angles, cover plates and web sheets shall be as long as practicable to avoid splicing. . . .

47. The pitch of rivets will not be less than 3 diameters, nor more than 6 inches (see ¶ 92), nor more than 16 times the thickness of the thinnest outside plate. No rivet will have a longer grip than 5 times its diameter, nor be nearer the edge of the metal through which it passes than $1\frac{1}{4}$ inches when the edges are machine or roll finished, and will not be nearer than $1\frac{1}{2}$ inches to a sheared edge.

51. Work will be designed in clean, handsome lines. In addition to the value and usefulness of members in the structure, they will be neatly finished.

### *Rivets.*

67. For main members $\frac{7}{8}$-inch rivets will preferably be used, with $\frac{3}{4}$-inch rivets for lateral members.

69. Whenever the grip length of rivet exceeds $2\frac{1}{2}$ inches, power-driven rivets will be insisted upon.

71. The dies will not exceed the diameter of rivet by more than $\frac{1}{16}$ inch.

### *Plate Girders.*

76. The webs of all plate girders will be of steel. . . .

77. No rivet holes will be pitched nearer than $1\frac{1}{2}$ inches to a sheared edge, or nearer than $1\frac{1}{4}$ inches to a roll-finished or machined edge of the webs.

78. Web splices will be made by two universal steel plates of the same thickness as the web plate. Stiffeners will be used at all web splices. The width of the splice plates shall be sufficient to admit the requisite number of rivets and to receive the stiffeners.

79. Wherever the unsupported distance between the flange angles exceeds 50 times the thickness of the web sheet, vertical stiffeners of angle iron shall be placed on each side of the girder. Stiffeners will be symmetrically spaced from the center of the girder, and the distance be-

tween them, center to center of rivets, will not be greater than the distance between centers of flange angles. If unequally spaced, the distance between them will decrease toward the ends.

80. There will be a pair of stiffeners at each end of all bed plates.

81. All stiffeners will have fillers under them of the same thickness as flange angles and as wide as stiffener angles.

82. The net section of tension flanges will be reckoned as the minimum section square across the flange, and the net section on any diagonal or broken line through two or more rivet holes must have 25 per cent excess.

83. In calculating shearing and bearing strains on web rivets of plate girders, the maximum shear acting on the outer side *MM* of any panel will be considered to be transferred to the flange angles in a distance *MO* (equals *MM*), and the number of rivets in the stiffener *MM* will follow the same rule.

84. All stiffeners, fillers and splice plates on the webs of girders must fit at their ends to the flange angles sufficiently close to be sealed, when painted, against admission of water, but need not be tool-finished.

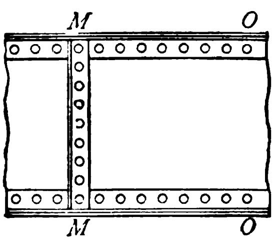



FIG. 20.

85. Web plates of all girders must be arranged so as not to project beyond the faces of the flange angles, nor on the top be more than $\frac{1}{16}$ inch below the face of these angles at any point.

86. To provide for local shear of heavy wheel loads, the rivet spacing in top flanges of deck-plate girders and stringers will not exceed 3 inches pitch when there are no cover plates, or 4 inches with cover plates.

87. The compression flanges of girders will be stayed at intervals not exceeding 15 times their width.

88. In through spans, stiffened gussets will run from top flange to each floor beam.

89. In deck spans the lateral bracing will extend from end to end, and no brace will make an angle less than 40 degrees with the girders, excepting end braces of skew spans.

90. All girders having flange plates will have one plate in each flange extending from end to end; and, with the exception of floor beams, girders will preferably have at least one flange plate.

91. When two or more plates are used on the flanges they shall either

be of equal thickness or shall decrease in thickness outward from the angles. and shall be of such lengths as to allow of at least two rows of rivets of the regular pitch being placed at each end of the plate beyond the theoretical point required.

92. When two or more cover plates over twelve inches wide are used in the flanges of plate girders, an extra line of rivets shall be driven along each edge to draw the plates together and to prevent the entrance of water. Plates over 17 inches wide will have three rows of rivets with 9 inches pitch for the outer row.

93. All joints in flanges, whether in tension or compression members, must be fully spliced, as no reliance will be placed upon abutting joints. The ends, however, must be dressed straight and true, so that there shall be no open joints.

94. Flange angles must be spliced with angle covers wherever cut within the length of the girder.

95. Splices must break joints with each other, one piece only being spliced at any point.

96. Cross-frames will be used at the masonry ends of all girders and at intermediate points when wind or centrifugal force makes it desirable.

97. All cross-frames will be made of angles and plates, and will be stiff rectangles of four members, viz., top, bottom and two diagonals.

98. Girders will be neatly finished at the ends. They will have a plate corresponding in width with the cover plates riveted to end stiffeners, and a corner cover at the top riveted to both top and end plates.

*Long Plate Girders.*

109. Plate girders of lengths from 75 feet to 100 feet may be built of medium steel, under special conditions, as follows:

110. All rivet holes in the angles and plates in both flanges will be drilled in the solid. Occasional small holes may be punched for purposes of bolting up and reamed afterward.

111. All other rivet holes in web plates, stiffeners, lateral braces and fillers may be punched of full size for riveting, provided the metal does not exceed $\frac{9}{16}$ inch thickness.

113. All web splices will have four rows of rivets, the middle rows being pitched five inches between centers.

114. Rivets will be of soft steel and power-driven whenever practicable.

115. The general requirements given under plate girders will apply to these also, except as modified above. See ¶¶ 77 to 98 inclusive.

*Shoes, Bed Plates, etc.*

155. There must be a pier box or plate of approved form under pedestal shoes at both ends, of sufficient depth to distribute the weight properly on masonry. These boxes or plates must be at least $\frac{3}{4}$ inch thick, must have planed surfaces and be of such dimensions that the greatest pressure upon the masonry will not exceed 250 pounds per square inch, and sheet lead not less than $\frac{1}{8}$ inch thick shall be interposed between them and the masonry.

156. Where two spans rest upon the same masonry a continuous plate not less than $\frac{3}{4}$ inch thick shall extend under the two adjacent bearings.

157. All the bed plates and bearings under fixed and roller ends must be fox-bolted to the masonry; for trusses, these bolts must not be less than $1\frac{1}{4}$ inches diameter; for plate and other girders, not less than $\frac{7}{8}$ inch diameter. The contractor must furnish all bolts, drill all holes, and set bolts to place with sulphur.

158. All bridges over 75 feet span shall have at one end nests of turned friction rollers, formed of wrought steel, running between planed surfaces. The rollers shall not be less than $2\frac{1}{2}$ inches diameter, and shall be so proportioned that the pressure per lineal inch of roller shall not exceed the product of the square root of the diameter of the roller in inches multiplied by 500 pounds ($500\sqrt{d}$). Bridges less than 75 feet span will be secured at one end to the masonry, and the other end shall be free to move by sliding upon planed surfaces.

159. Friction rollers must be so arranged as to be readily cleaned and to retain no water.

160. While the roller ends of all trusses must be free to move longitudinally under change of temperature, they shall be anchored against lifting or moving sideways.

In view of the statement in ¶ 18 above it will be specified that the live load shall consist of two coupled Lehigh Valley

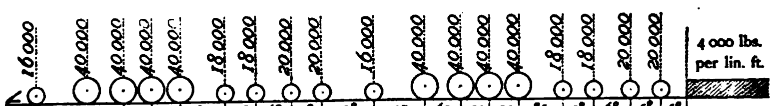



FIG. 21.

typical consolidation locomotives and train as shown in Fig. 21. The numbers above the wheels show their weights in pounds for both rails of a single track and the numbers between them give their distances apart in inches.

The general drawing of the complete structure is shown on Plate III, to which reference may be made as the design proceeds without further attention being called to it in the text.

## ART. 31. LIVE-LOAD STRESSES.

The following stresses due to the load specified in the preceding Article were obtained graphically and refer to one girder.

| Distance of Section from Support. | Wheel at the Section. | Maximum Bending Moment. | Wheel at the Section. | Maximum Vertical Shear. |
|---|---|---|---|---|
| Feet. | No. | Pound-feet. | No. | Pounds. |
| 0 | — | 0 | 2 | 119 000 |
| 5 | 2 | 530 000 | 2 | 105 600 |
| 10 | 3 | 962 000 | 2 | 92 600 |
| 15 | 3 | 1 315 000 | 2 | 80 000 |
| 20 | 4 | 1 580 000 | 2 | 68 800 |
| 25 | 12 | 1 780 000 | 2 | 58 900 |
| 30 | 13 | 1 927 000 | 2 | 50 100 |
| 35 | 13 | 2 022 000 | 2 | 41 800 |
| 40 | 13 | 2 045 000 | 2 | 33 900 |

The diagram employed also contained the equilibrium polygon for a train following the first locomotive and indicated at a glance that this load will produce slightly larger shears at sections 15, 20, 25 and 30 than the one specified, the values being 80 400, 69 800, 59 800 and 50 400 pounds respectively.

The absolute maximum bending moment occurs at the middle, which is rather unusual for a girder. The moment remains almost the same for a foot each way and is 2 040 000 pound-feet at a distance of two feet from the middle.

The following table contains the simultaneous bending moments in each pair of adjacent sections when the load is so placed as to produce the maximum vertical shear in the section nearer the middle of the girder. The moments are

| Distance of Section from Support. | Load in Position for Maximum Shear at Section. | | | |
|---|---|---|---|---|
| | 5′ | 10′ | 15′ | 20′ |
| Feet. | | | | |
| 0 | 0 | | | |
| 5 | 530 000 | 480 000 | | |
| 10 | | 945 000 | 850 000 | |
| 15 | | | 1 255 000 | 1 125 000 |
| 20 | | | | 1 470 000 |
| Difference.. | 530 000 | 465 000 | 405 000 | 345 000 |
| | 25′ | 30′ | 35′ | 40′ |
| 20 | 1 315 000 | | | |
| 25 | 1 610 000 | 1 430 000 | | |
| 30 | | 1 680 000 | 1 470 000 | |
| 35 | | | 1 680 000 | 1 445 000 |
| 40 | | | | 1 610 000 |
| Difference.. | 295 000 | 250 000 | 210 000 | 165 000 |

expressed in pound-feet. Their differences are proportional to the shear transmitted from the flanges to the web, und these will be used in determining the rivet spacing in the flanges, as will be explained in Art. 39.

The method of finding the moments and shears in a plate girder under locomotive and train loads is given in Part II, Art. 40, but a modification of it somewhat more advantageous will be employed in this problem, the description of which will now be given.

The criterion for the position of the wheel loads which produces the maximum moment in any given section of a girder is the same as that deduced for trusses in Part I, Art. 61, and is

$$P' = \frac{l'}{l} W,$$

in which $W$ is the whole load on the bridge, $P'$ the part of the load on the left of the section, $l'$ the distance from the section to the left support, and $l$ the span. To use this criterion let a "load line" $ABCD$, composed of a series of steps, be con-

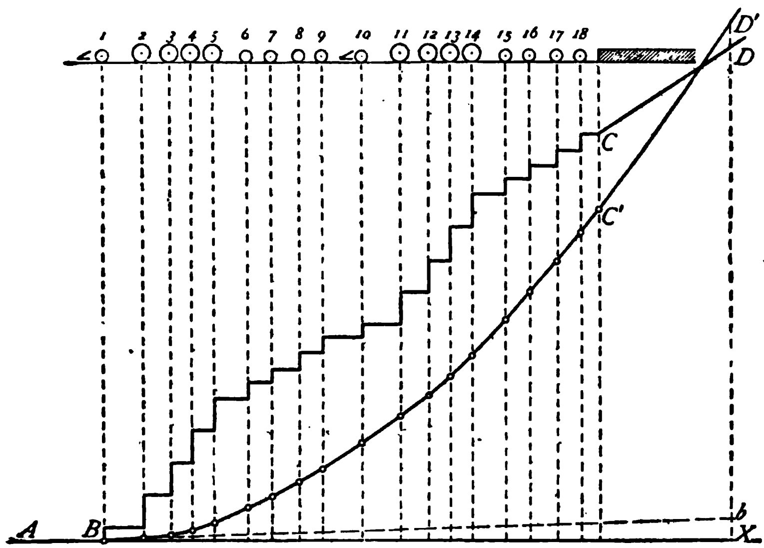



FIG. 22.

structed as shown in Fig. 22. The rise in each step indicates the weight and the position of the corresponding wheel, suitable scales being used for the distances and weights. Any ordinate to this load line gives the sum of all the loads on its left, and as the diagram is drawn on profile paper its value can be read off directly.

The equilibrium polygon $ABC'D'$, consisting of a broken line for the locomotive and a part of a parabola for the train, was drawn in this case by laying off the moment ordinates under each wheel as taken from a tabulation diagram on hand similar to the one shown in Part I, Art. 62.

On a sheet of tracing paper let the span *ab* of the girder be laid off to the same linear scale as that shown by the profile paper, the half span divided into the required number (8) of equal parts and indefinite ordinates erected at these points. Fig. 23 shows the completed diagram placed in position on the load line.

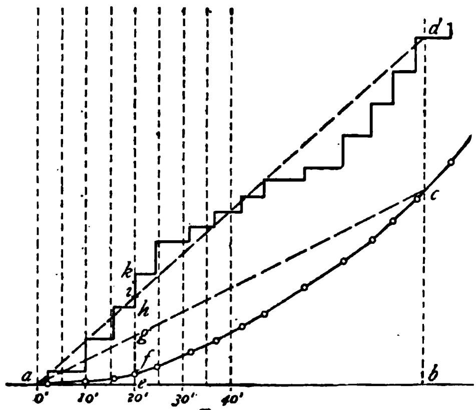



FIG. 23.

It is very important that when the base lines of the two diagrams coincide their ordinates should be truly parallel. The sections are five feet apart and the 20-foot section is made to coincide with wheel 4. If this be the proper position for the maximum moment in the 20-foot section, the equation $P' = \frac{l'}{l} W$ must be satisfied. Remembering that the horizontal axis on the left of *a* is to be considered as a part of the load line, connect by a straight line the points *a* and *d* where the ordinates at the supports intersect the load line. Now the ordinate *bd* equals *W*, *ab* equals *l*, *ae* equals *l'*, and hence $W\frac{l'}{l}$ is represented by the ordinate *ei*. If the wheel 4 is just on the right of the section the ordinate *eh* represents *P'*, if just on the left the ordinate *ek* is the load *P'*; and when it is at the section the load *P'* may be regarded as having any value between these limits. The condition is therefore satisfied, and this position will give the maximum moment at section 20'. All the possible positions for all the sections can be determined in a few minutes, by using a small silk thread, shifting the tracing paper in each case so as to bring a wheel over the section, stretching the thread as indicated above and noting whether it intersects that wheel on the load line.

The value of the maximum bending moment at section 20′ may now be obtained by drawing the closing line and measuring the ordinate *fg*. A scale consisting of a separate strip of profile paper cut from the same sheet is convenient for this purpose. If the left end *a* of the closing line were always on the axis *ab* greater precision would be attained by reading the moment *bc*, multiplying it by the ratio $l' \div l$, which in this example is one fourth, and subtracting the moment *ef*, which is known and usually marked on the diagram. In plate girders, however, *a* is frequently not on the axis. In the girder whose moments are given in this article the positions satisfying the criterion for the various sections were found to be as follows:

| Section. | Wheel at Section. | Section. | Wheel at Section. |
|---|---|---|---|
| 5′ | 2, 3 | 25′ | 4, 12, 13 |
| 10′ | 2, 3, 4 | 30′ | 12, 13 |
| 15′ | 3, 4 | 35′ | 12, 13 |
| 20′ | 4, 12 | 40′ | 4, 5, 8, 9, 10, 12, 13 |

It is therefore better to make the scale large enough to insure the requisite precision when the ordinates are read off directly by a separate scale so that only one value needs to be recorded. Where more than one position satisfies the condition the use of the dividers will show which is the largest, and its value alone needs to be carefully read and recorded. In order to enable a large vertical scale to be used on the width of an ordinary sheet of profile paper, the equilibrium polygon, or moment diagram, was consolidated as shown in Fig. 24. In this way the vertical divisions on profile paper, plate B, which measure a little more than 0.8 inch, could be taken as 500 thousand pound-feet instead of 1000 thousand pound-feet as before. The linear scale used was 4 feet to an inch, the actual working diagrams being therefore about ten times as large as those here given.

By reference to the above table of positions it is seen that the ordinates to be measured lie in the left half of the diagram so that acute intersections of the ordinates with the polygon are avoided. If this diagram were not also to be used for obtaining shears a further improvement would be made by inclining the axis

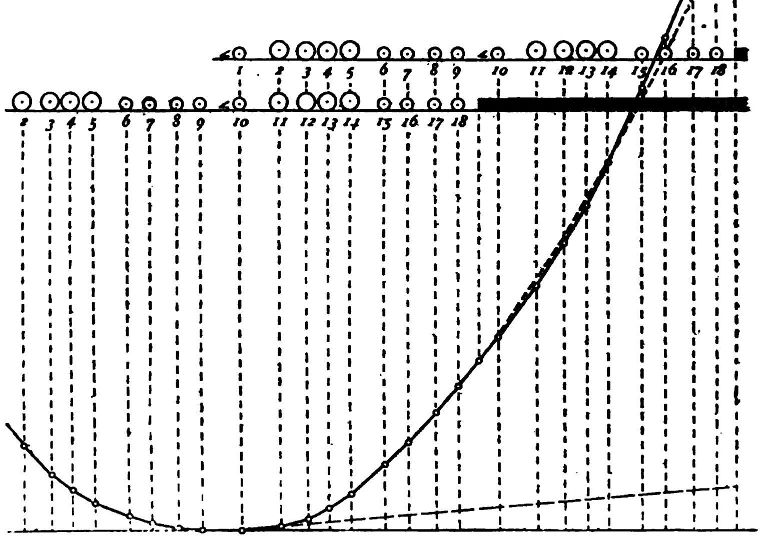


FIG. 24.

downward towards the right, thus bringing the closing lines more nearly horizontal.

Before finding the maximum vertical shears at the various sections it is necessary to know the position of the live load causing them. As shown in Part II, Art. 40, the maximum shear in any section occurs when one of the loads at the head of the locomotive is at the section. If $M'$ be the moment of all the loads about the right support when wheel 1 is at the section, and $M''$ when wheel 2 is at the section, the corresponding values of the vertical shear are

$$V_1 = \frac{M'}{l}, \quad \text{and} \quad V_2 = \frac{M''}{l} - P_1,$$

$P_1$ being the weight of wheel 1. There will be some section where the shear due to both positions will be equal. Equating these values and transposing,

$$M'' - M' = P_1 l.$$

In Fig. 25 let *cd* be the moment $M''$ and *ab* the moment $M'$. The distance *ac* between these moment ordinates is equal to

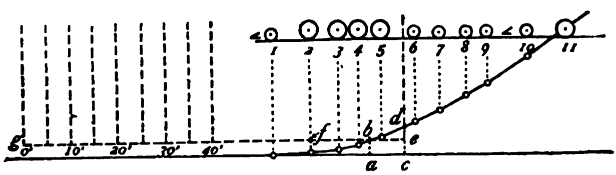



FIG. 25.

the distance between the wheels 1 and 2. If $V_1 = V_2$, $de = cd - ab = M'' - M' = P_1 l$. The position of the section where $V_1 = V_2$ may then be found as follows: Place the girder diagram (constructed on the tracing paper) on the moment diagram (Fig. 22) with its left support at wheel 1 and mark on its section or ordinate at the right support the distance to the line *Bb*, which is the side of the equilibrium polygon on the right of wheel 1 produced. This is $P_1 l$ (Part II, Art. 7). Move the former diagram to the left until the section at the right support is at wheel 2 and mark the position of wheel 1. The two points marked are shown in Fig. 25 at *d* and *b* respectively. Now move the tracing with the point *d* remaining on the equilibrium polygon and with its axis horizontal until a position is reached where *b* is also on the polygon. Mark the position of wheel 2 at *f*. It is easily remembered what wheel is to be marked by noticing that the right-hand ordinate *cd* is $M''$, which by the notation is the moment at the support when wheel 2 is at the section. At every section, therefore, between $f$ and $e$ the greatest shear will be produced when wheel 1 is at the section, and for all sections between $f$ and $g$ (the left support) when wheel 2 is at the section.

In the girder under consideration the shears under the first two wheels are equal at a section a little over 60 feet from the left support. For the live load here employed no plate girder has yet been built in which wheel 3 at any section gives a greater shear than wheel 2.

When the live load consists of passenger locomotives it is possible that for some spans the section in which $V_2 = V_3$ may be on the right of that where $V_1 = V_2$, in which event it is necessary to find where $V_1 = V_3$. No position for maximum shear then requires wheel 2 to be at the section.

The reactions at the left support are now obtained by dividing the moment $M''$ by the span. If the first wheel is off the girder the ordinate below the line $Bb$ in Fig. 22 must be deducted from $M''$, and the remainder divided by the span.

Very few lines need to be drawn for any particular problem, and it will be observed that these few are all on the tracing paper. The moment diagram may be used for other girders or trusses until wear interferes with its accuracy. The graphical method of determining position by means of a load line was first published by WARD BALDWIN in Engineering News, Vol. XXII, pp. 295, 345, and 615 (letter dated Dec. 11, 1889).

## ART. 32. THE TRACK.

The specifications fix the dimensions and spacing of the cross-ties and guard-rails to be used. In order to determine the unit stress in the cross-ties assume rails weighing 80 pounds per yard, their height being 5 inches, the width of head $2\frac{1}{2}$ inches and width of base 5 inches. Each driver transmits a weight of 20 000 pounds to the rail, which distributes it over more than one cross-tie, but the weight carried by each cross-tie cannot be accurately computed since the rail deflects slightly and the cross-tie also yields under the pressure as the load passes over it. It may be assumed that the maximum weight on each cross-tie is approximately one half of the load on one driver. The rails are $4' \ 8\frac{1}{2}'' + 2\frac{1}{2}'' = 4' \ 11''$ apart between centers, and the girders are 7 feet apart according to ¶ 4 of the specifications. The cross-tie is then a beam bearing two concentrated loads, of 10 000 pounds each, 4′ 11″ apart and equidistant from the center, its span being 7 feet. Since the cross-tie is notched $\frac{3}{4}$ inch over the girder and the edge of this notch will be but a few inches distant from a vertical through the outer edge of the base of the rail, the effective depth will be taken at $9\frac{1}{4}$ inches. The unit stress in the outer fiber will be found to be 1095 pounds per square inch due to the wheel loads alone, the weight of the cross-tie and rails increasing this by less than 20 pounds. For southern yellow-pine timber, which is preferably employed for bridge cross-ties, this is below the safe value, usually taken at 1200 pounds.

In order that the upper surfaces of the cross-ties shall be in the same horizontal plane it will be necessary to increase the depth of the notches, as additional cover plates are met in passing toward the middle of the girders. As the combined thickness of these cover plates at the center is not yet determined the depth of cross-ties could not now be computed if it were to be based on the unit stress instead of being fixed by the specifications. In order to observe, however, the increase of stress due to deepening the notches, suppose that at the center the notches must be $2\frac{1}{2}$ inches; the unit stress will then be found to be 1675 pounds per square inch without considering the weights of rails and cross-tie. If this be regarded as excessive all the cross-ties must be made 12 inches deep.

## ART. 33. LOADING.

(See paragraphs 18–20, 22, 23, 25 and 26 of the Specifications in Art. 30.)

Before determining the dead and wind loads it is well to fix the depth of the girder. Approximate theoretic investigations as well as a comparison of complete designs show that the economic depth varies from one eighth to one twelfth of the span, the ratio decreasing as the span increases. As shown in Art. 10, increasing the depth for any given span reduces the weight of the flanges but at the same time increases the weight of the web and its stiffeners, and *vice versa*. Different specifications will produce different economic depths for the same span and live load, while small variations from the economic depth do not materially affect the weight. In the present example the depth of web will be assumed as one eleventh of the span or 87 inches = 7 feet 3 inches, and after the design is completed the actual economic depth will be computed.

To have some guide in assuming the dead load for one girder a preliminary estimate was made in which only the sections of its principal members were roughly computed. The weight of the lateral systems was approximated by taking the corresponding known weight for a girder whose span differed very considerably from 80 feet and applying a correction. This estimate made the weight of one girder 43 400 pounds and that of one half of both lateral systems 3100 pounds. Hence the weight of both items is taken at $7.25l$ pounds per linear foot, $l$ being the length of the span in feet, and which gives $7.25 \times 80 \times 80 = 46\,400$ pounds. The weight of the track (¶ 20) for one girder $= \frac{1}{2} \times 400 \times 80 = 16\,000$ pounds. The total dead load is then

$$46\,400 + 16\,000 = 62\,400 \text{ pounds.}$$

The pressure of the wind on the side of the train and of the girder and track will cause a transfer of load from the windward to the leeward girder. The decrease of load in the windward girder and the equal increase of load in the leeward girder form a couple whose lever arm is the distance between the girders and whose moment is equal to the overturning moment of the wind, the horizontal axis of rotation being in the upper surface of the expansion rollers which support the girders. The center of pressure on the train is about 7 feet above the rails. The vertical distance from the top of the rail to the top of the rollers is equal to the sum of the following distances: height of rail, net depth of cross-tie, thickness of four cover plates, depth of web, thickness of one cover plate, and thickness of two sole, or bearing, plates. This distance is estimated to be $5'' + 9\frac{1}{4}'' + 2\frac{3}{4}'' + 87'' + \frac{3}{4}'' + 1\frac{3}{4}'' = 8'\ 10\frac{1}{2}''$, making the lever arm of the wind pressure on the train $15'\ 10\frac{1}{2}''$, or say 16 feet. Hence the overturning moment of the wind on the train transfers to the leeward girder a uniformly distributed load of

$$80 \times 300 \times 16 \div 7 = 54\,860 \text{ pounds.}$$

Paragraph 22 of the specifications does not refer to girders, but a reference to the succeeding sections indicates that the wind pressure is taken at 30 pounds per square foot of surface exposed. The average depth of the girder and track is estimated at 100 inches, the centre of pressure being about 51 inches above the support. The overturning effect of the wind pressure on the girder therefore transfers to the leeward girder a uniform load of

$$80 \times \tfrac{100}{12} \times 30 \times \tfrac{51}{12} \div 7 = 12\,140 \text{ pounds.}$$

The total transfer of load due to the wind is thus seen to exceed the dead load.

Only the area of one girder and the side elevation of the track were considered above as exposed to the maximum wind pressure. Where the spacing of the girders is less than their depth and the floor system covers about 60 per cent of the horizontal area included between its limits, as in the above example, the leeward girder may be regarded as almost entirely protected. When the depth is less than the spacing it will be better to assume that a portion of the surface of the leeward girder is also exposed to the wind pressure, such portion not to exceed about 50 per cent of the vertical projection of the girder. In some localities it may be advisable to extend this to the case of very deep girders.

## ART. 34. SECTIONAL AREA OF FLANGES.

(See paragraphs 29, 30, 31 *c* 31 *j*, 32 and 35 of the Specifications in Art. 30)

The maximum bending moment (¶ 29) due to dead load is

$$\tfrac{1}{8} \times 62\,400 \times 80 = 624\,000 \text{ pound-feet.}$$

Similarly, the moments due to the overturning effect of the wind on the train and on the girder are respectively 548 600 and 121 400 pound-feet.

The wind pressure also causes a bending moment in the lateral systems of which the flanges form a part. In order to find the maximum stresses in the flanges due to this cause, it will be near enough to disregard the form of the lateral webbing and to use the moment at the middle due to the wind pressure as a uniform load. Of the 100 inches average depth of girder and track, about 54 inches belong to the upper system and 46 to the lower. For the upper system the uniform load is

$$(80 \times 300) + (80 \times \tfrac{54}{12} \times 30) = 34\,800 \text{ pounds,}$$

and for the lower system

$$80 \times \tfrac{46}{12} \times 30 = 9200 \text{ pounds.}$$

The corresponding bending moments are 348 000 and 92 000 pound-feet. The maximum live-load bending moment is 2 045 000 pound-feet. (Art. 31.)

Before finding the flange stresses it is necessary to know the effective depth (¶ 30), but as this is generally the distance between the centers of gravity of the flanges its exact value cannot be obtained until the section of the flange is designed. As the depth used must not exceed the distance out to out of flange angles, the depth of the web, which is 87 inches, will be taken, and probably the required distance will not differ much from this, since several cover plates will be required at the center of the girder.

Reducing the several moments to pound-inches and dividing by 87 inches the following flange stresses are obtained:

| | | | |
|---|---|---|---|
| Dead load | | 86 070 | pounds. |
| Live load | | 282 070 | " |
| Overturning effect of wind | on train | 75 670 | " |
| | on girder | 16 750 | " |

The flange stresses due to the moments in the lateral systems are found in a similar manner, dividing, however, by 84 inches instead of 87 inches.

| | | |
|---|---|---|
| In upper lateral system | 49 710 | pounds. |
| In lower lateral system | 13 140 | " |

It is now necessary to see whether the flange stresses due to the wind are to be taken into account in determining the area of the section (¶ 32). Since the compression flange of a plate girder is to have the same cross-section as the tension flange (¶ 31*j*), the stresses in the latter will be considered first. The maximum stresses due to wind aggregate $75\,670 + 16\,750 + 13\,140 = 105\,560$ pounds. The dead- and live-load stresses amount to 368 140 pounds, 25 per cent of which is 92 035 pounds, which being less than 105 560 pounds requires the wind stresses to be taken into account, the unit stress being increased 25 per cent.

The unit stress (¶ 31*c*) for medium steel is $9000\left(1 + \frac{\text{min.}}{\text{max.}}\right)$

pounds per square inch. The minimum stress is 86 070 − (16 750 + 13 140) = 56 180 pounds, and the maximum is

$$86\,070 + 282\,070 + 75\,670 + 16\,750 + 13\,140 = 473\,700 \text{ lbs.}$$

Then

$$9000\left(1 + \frac{56\,180}{473\,700}\right) = 10\,062,$$

which being increased by 25 per cent gives a unit stress of 12 580 pounds per square inch. The area of the flange section is therefore 473 700 ÷ 12 580 = 37.65 square inches.

## ART. 35. COMPOSITION OF THE FLANGES.

(See paragraphs 30, 31*j*, 35, 82, 85, 86 and 92 of the Specifications in Art. 30.)

The area of 37.65 square inches found in the preceding article is the net section of the flange in tension, ¶ 35 requiring that the web shall not be considered as sustaining any part of the bending moment. Paragraph 31*j* states that the compression flange shall have the same cross-section as the tension flange. The flange is usually composed of two angles and one or more cover plates. Before selecting these parts of suitable size, let ¶¶ 82 and 36 be consulted to see what deductions must be made for rivet holes. Seven-eighth-inch rivets will be used (¶ 67), hence each rivet hole to be deducted is to be taken as one inch in diameter.

Where the area is so large it is desirable to use large angles. Double rows of rivets will be required in both legs of the angles. The $6 \times 6 \times \frac{7}{8}$ inch angle is the largest manufactured (Pocket Companion, page 38), but the $6 \times 6 \times \frac{3}{4}$ inch angle has a finishing pass and will be selected.

As the Pocket Companion (see page 48) does not give the position of the pitch lines in angles for double rows of rivets, the Pencoyd standard will be followed in this particular (see Art. 15). The same standard will be used hereafter without further reference, since it makes a distinction between angles used as flanges and as braces. The maximum allowable size of rivet will, however, be taken from the table in the Pocket Companion.

Fig. 26 shows the standard position of the pitch lines for this size angle in flanges, while Fig. 27 shows a longitudinal section at mid-thickness of this angle when flattened out, $a$ being 2.25 inches and $b$ 4.15 inches. Paragraph 82 requires that only two rivet holes shall be deducted from the gross section of each angle, unless the sum of the net distances $e + f + e$ exceeds $c + d$ by less than 25 per cent. The pitch of the rivets cannot now be determined, and must therefore be assumed, subject to later revision if necessary. The maximum allowable pitch referred to in ¶ 86 refers to the longitudinal distance between successive rivets whether in the same or in adjacent rows, so that if there are two rows of rivets in each leg of the angles the maximum allowable distance between successive rivets in the same row is 8 inches. As the flanges will require to be spliced near the middle of the girder and the length of the splice is directly pro-

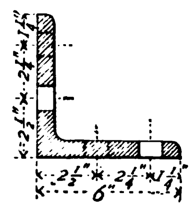


FIG. 26.

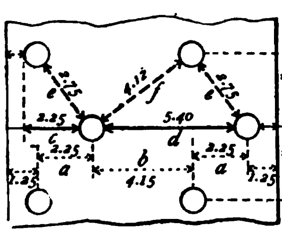


FIG. 27.

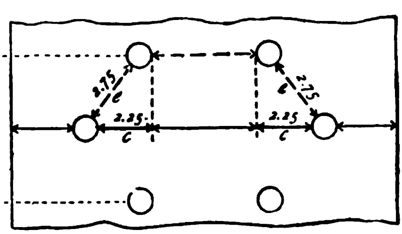


FIG. 28.

portional to the rivet pitch, it will be assumed that the maximum pitch in each row does not exceed 6 inches. The distance $c = a = 2.25$ inches, $d = 5.40$ inches, and for a pitch of 6 inches $e$ is found to be 2.75 and $f$ to be 4.12 inches. Since $2.75 + 4.12 + 2.75 = 9.62$ and $(2.25 + 5.40)1.25 = 9.56$, only two rivet holes need to be deducted from the sectional area of each angle.

Fig. 28 shows a piece of a cover plate, and in this case ¶ 82 requires that $e+e$ shall be 25 per cent greater than $c+c$. As $c$ and $e$ have the same values as in Fig. 27 the excess is found to be less than 25 per cent. The equivalent distance square across when $e=2.75$ inches is 80 per cent of this distance, or 2.20 inches, and for $e+e$ is 4.40 inches, which subtracted from $c+c=4.50$ inches gives 0.10 inch; therefore to obtain the net section of the cover plates 2.1 rivet holes must be deducted when the rivet pitch is 6 inches.

The next step is to consider the width of the cover plates. The first sentence in ¶ 92 seems to refer to flanges having only one row of rivets in the angles, since it is not customary to put an extra row outside of the angles unless the plates extend at least 3 inches beyond them. Since plates over 17 inches wide require three rows of rivets on each side such a width will be avoided unless more than three cover plates of practicable thickness would be required. In shorter spans the thickness of wrought-iron cover plates is usually limited to one-half inch on account of punching, but in this case (¶¶ 109, 110) these limitations do not apply, and ¶ 111 allows metal outside of the flanges to be punched up to a thickness of $\frac{9}{16}$ inch. A width of 16 inches will therefore be selected for the plates, this width requiring two rows of rivets on each side and extending less than 2 inches beyond the angles.

The following composition of the flange section will furnish the required area (Pocket Companion, pages 106, 192):

| | | | |
|---|---|---|---|
| 2 angles, | $6''\times6''\times\frac{3}{4}''$, | $2(8.44-1.50)$ | $=13.88$ sq. in. |
| 1 cover plate, | $16''\times\frac{5}{8}''$, | $(16.00-2.10)\frac{5}{8}$ | $=8.69$ |
| 2 cover plates, | $16''\times\frac{9}{16}''$, | $2(16.00-2.10)\frac{9}{16}$ | $=15.64$ |
| | | Total net section | $=38.21$ sq. in. |

The effective depth (¶ 30) can now be computed and compared with the value assumed. In the following computations the plane of reference is in each case the plane of the horizontal backs of the flange angles. In Pocket Companion, page 105, the distance of the center of gravity of a $6 \times 6$ inch angle from its back is given as 1.66 inches for a thickness of $\frac{7}{16}$ inch and 1.82 inches for a thickness of $\frac{7}{8}$ inch, and by interpolation, which is sufficiently precise, the value for a thickness of $\frac{3}{4}$ inch is 1.77 inches. For the gross area of the upper flange section:

| | | | |
|---|---|---|---|
| 2 angles, | $6'' \times 6'' \times \frac{3}{4}''$, | $16.88 \times 1.77$ | $= 29.88$ |
| 1 plate, | $16'' \times \frac{5}{8}''$, | $10.00$ | |
| 2 plates, | $16'' \times \frac{9}{16}''$, | $18.00 — 28.00 \times 0.875$ | $= 24.50$ |
| | | 44.88 sq. in. | 5.38 |

The center of gravity of the upper flange is therefore $5.38 \div 44.88 = 0.12$ inch below the backs of the angles. For the net lower flange section:

| | | |
|---|---|---|
| 2 angles, $6'' \times 6'' \times \frac{3}{4}''$, | $16.88 \times 1.77$ | $= 29.88$ |
| Less 4 rivet holes, | $3.00 \left( \frac{4.75 + 2.5 + 2 \times 0.375}{4} \right)$ | $= 6.00$ |
| | 13.88 sq. in. | 23.88 |
| 1 plate, $(16'' - 2.1'')\frac{5}{8}''$ | | |
| 2 pl., $2(16'' - 2.1'')\frac{9}{16}''$ | $— 24.33 \times 0.875$ | $= 21.29$ |
| | 38.21 sq. in. | 2.59 |

The center of gravity of the lower flange is $2.59 \div 38.21 = 0.07$ inch above the backs of the angles. In the preceding computation the moment of the area of four rivet holes in one angle and located in two sections 3 inches apart was deducted instead of the moment for the rivet holes in both angles and lying in the same section. If the latter method had been adopted the result would have been 0.03 and 0.11 inch for the two successive sections. The preceding value is the mean of these two. As the actual sections of these various shapes are subject to a slight variation and their surfaces cannot be brought into mathematical contact the effective depth is computed only to the nearest tenth of an inch. In this example the depth equals $87.0 - (0.12 + 0.07) = 86.8$ inches, provided the horizontal backs of the angles are placed even with the edges of the web. Using this value of the effective depth the revised stresses are

| | | | |
|---|---|---|---|
| Dead load | | 86 270 | pounds |
| Live load | | 282 720 | " |
| Overturning effect of wind | on train | 75 840 | " |
| | on girder | 16 780 | " |

while those in the lateral system remained unchanged. The maximum stress is 474 750 and the minimum 56 350 pounds. The unit stress is 12 590 pounds per square inch, and the revised area of the flange section $474\,750 \div 12\,590 = 37.71$ square inches, which shows that the section chosen needs no revision.

By comparing the final value of the area with the provisional value the student may gain some idea as to the relative effect of small changes in the effective depth or in other items affecting it, and thus learn what degrees of precision are required in the various computations.

In some cases it may be advantageous to consider ¶ 85 before fixing the effective depth.

## ART. 36. WEB SECTION.

(See paragraphs 31 *l*, 35, 42 and 76 of the Specifications in Art 30.)

The live-load vertical shears as given in Art. 31 are laid off as ordinates in Fig. 29 above the axis $AB$, giving the curve $FG$, to show the variation in shear from the support to the middle of the girder. The shear due to the dead load, and the overturning effect of the wind on the train and girder is (62 400 + 54 860 + 12 140) ÷ 2 = 64 700 pounds. (Art. 33.)

Assume for convenience that the greatest shear due to the overturning effect of the wind on the moving train decreases

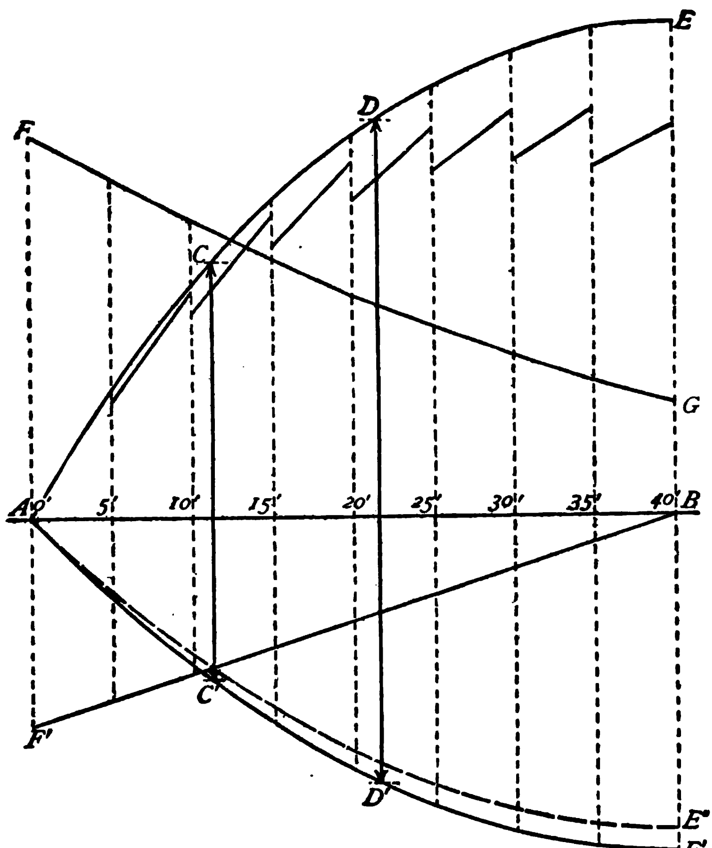



FIG. 29.

uniformly to zero at the middle of the girder similarly to the dead load; then $F'B$ is the shear line for these loads if $AF'$ is laid off equal to 64 700 pounds. The maximum shears at the successive sections are then as follows: at 0′, 183 700; at 5′, 162 200; at 10′, 141 100; at 15′, 120 400; at 20′, 101 200;

at 25′, 83 200; at 30′, 66 300; at 35′, 49 900; and at 40′, 35 100 pounds.

The web of a plate girder consists of one or more plates stiffened, at intervals not exceeding the depth of the girder, by means of angle irons riveted to it. Generally the thickness of these plates is uniform throughout, but sometimes the heavy shear towards the ends is provided for by increasing their thickness, or by re-inforcing the web by additional plates.

The web is to be proportioned to take the entire vertical shear (¶ 35), the unit stress being 6000 pounds per square inch (¶ 31 *l*). At the support this will require a net section of 183 700 ÷ 6000 = 30.62 square inches. According to ¶ 42 no plate is to be less than $\frac{3}{8}''$ thick, and web plates are rarely made to exceed $\frac{1}{2}''$ in thickness. Since rivet holes in the web are punched $\frac{15}{16}$ inch in diameter (¶ 71), if the latter thickness be chosen it will have a gross section of $87 \times \frac{1}{2} = 43.5$ square inches and will allow 27 rivets to be placed in the same vertical row in the stiffeners at the inner end of the bearing plate, which will barely be sufficient.

At 5 feet from the support the maximum shear requires a net section of 162 200 ÷ 6000 = 27.03 square inches. If the web be taken $\frac{3}{8}''$ thick the gross section will have an area of $87 \times \frac{3}{8} = 32.62$ and will allow 15 rivets in a vertical row at that section (¶ 36). By using a $\frac{3}{8}''$ re-inforcing plate on each side of the web extending from the end to a short distance beyond the 5′ section, and if not more than 15 rivets, including the flange rivets in the web, be placed in the same row along the vertical edge of these re-inforcing plates, the conditions will be met. This arrangement will be adopted.

In Chaps. XII and XVIII will be found discussions and designs of plate-girder spans for railroad bridges, to which the student may refer for comparison.

## ART. 37. WEB SPLICES.

(See paragraphs 35, 46, 77, 78, 84 and 113 of the Specifications in Art. 30.)

The practicable length of the web sheets (¶ 46) depends not only on the maximum sizes of sheets manufactured, but also on the available facilities in the shops for handling them in the manufacture of girders. The location of the sections where the shears are found is governed by the length of sheets. The web will be spliced, in this example, at the end of each alternate 5-foot division. The thickness of the splice plate, one on each side, is the same as that of the web (¶ 78), or $\frac{3}{8}$ inches.

At least four rows of rivets will be required (¶ 113) in the splice plates, and they must be wide enough to receive the stiffeners. If the plates be taken 14 inches wide and each outer row of rivets placed $1\frac{3}{4}$ inches from the edge, the distance from an inner to an outer row will be $2\frac{3}{4}$ inches, since the inner rows must be 5 inches apart (¶ 113). If the smaller leg of the stiffeners be $3\frac{1}{2}$ inches the clearance between the edge of the stiffener and the rivet heads of the outer row is $2.75 - 1.75 - \frac{1}{2}(1.44) = 0.28$ inch, since the diameter of the rivet head is $1\frac{7}{16}$ inches (Art. 14) and the distance of the pitch line from the edge of the stiffener is $1\frac{3}{4}$ inches (Art. 15). This clearance is sufficient.

In order to find how many rivets are required in the splices, let the strength of a rivet be determined (¶ 36). The area of cross-section of a $\frac{7}{8}$-inch rivet is 0.6013 square inch (Pocket Companion, page 203). The unit shearing stress of soft-steel rivets (¶ 114) is 6600 pounds per square inch (¶ 31 *l*). The strength is then

in single shear, $0.6013 \times 6600 = 3970$ pounds,
and in double shear, $2 \times 3970 = 7940$ pounds.

The bearing area of the same rivet in a $\frac{3}{8}$-inch web plate is $\frac{7}{8} \times \frac{3}{8} = 0.3281$ square inch and the unit stress is 13 200 pounds (¶ 31 *m*), making the

$$\text{bearing strength} = 0.3281 \times 13\,200 = 4330 \text{ pounds.}$$

As the rivets in the web splices are in double shear their strength depends on the bearing value. At 10 feet from the support the splice requires $141\,100 \div 4330 = 32.6$ rivets in each web sheet for shearing alone, and if the rivets in the flanges are excluded this would require a pitch of 4.6 inches with the rivets in double rows. The following number of rivets would be required in each half of the remaining splices: at 20 feet, 23.4; at 30 feet, 15.3; and at 40 feet, 8.1.

Let an investigation now be made to determine what number of rivets would be required in order to make the splice as strong as the net section of the web, but before doing so it may be well to review some elementary principles.

According to the theory of stresses in beams, as developed in Mechanics of Materials, if the web of a plate girder is continuous the flanges receive their stresses by increments from the web through the flange rivets, these increments being largest at the end and steadily decreasing toward the middle, while the total flange stress reaches its maximum at or near the middle under the combined influence of fixed and moving loads. As the web is riveted to the flange and must elongate or shorten with it, it is evident that the web carries a part of the moment. This part is approximately equal to the flange unit stress multiplied by one sixth of the net area of the web, or about one eighth of its gross area.

Now suppose that the web be spliced on the left of the middle and that the rivet holes in the web splice are elongated horizontally so as to allow the web to freely change its length. The entire bending moment at the splice is now held in equilibrium by the resisting moment of the flange stress, or in other words the flange may be said to carry the entire moment. If on the right-hand side of the splice a portion of the web and the flange were not riveted together the lower flange would elongate under its stress, while the free web would keep its length unchanged, but when they are riveted together the rivets cause some of the flange stress to be transferred back to the web in their effort to equalize the extension of the adjacent fibers in the flange angles and web. As a result of this action some of the rivets near the splice will be overstrained. The farther the section is from the middle of the girder the greater will be this overstraining effect.

Let the splice with the elongated holes now be replaced by a regular splice containing only the number of rivets required for shearing as computed above. That portion of the direct tension which before was transferred by the flange rivets to the flange on the left of the splice and then transferred back on the right will now have some of it passing directly through the rivets in the splice and so overstraining them, or to state it differently the rivets in the splice will partly relieve the overstrained flange rivets on both sides of the splice by sharing with them this condition. Just what exact amount of stress each set of rivets carries it may not be possible to compute, but it is clear that more rivets will be required than the number computed under the supposition that the entire vertical shear is taken by the web alone.

The true stress in the rivets in any web splice is the resultant of the stress due to the vertical shear and that due to the part of the bending moment resisted by the web. As the shear at the center is nearly if not quite zero when the bending moment is a maximum, if the number of rivets be computed so as to be equally as strong as the net section of the web under flexure and the same number of rivets be placed in all the other splices the result will be on the side of safety, and one sixth of the net area of the web may also be considered as flange area. If, however, as in the specifications governing this design (¶ 35), the flanges shall be proportioned to resist the entire bending moment, then the safe number of rivets in the web splices will be intermediate between that depending on shear alone and that obtained by the method just described. The computation referred to above will now be made. Assuming a pitch of 3 inches, the rivets in the outer row of the spliced web sheet are at the following distances in inches from the neutral axis: 0, 3, 6, 9, 12, 15, 18, 21, 24, 27, 30, 33, 36, and 38.75, the last distance referring to the rivet in the flange angle. In the next row the distances are as follows: 1.5, 4.5, 7.5, 10.5, 13.5, 16.5, 19.5, 22.5, 25.5, 28.5, 32.25, 36, and 41, the last one being a flange rivet. If the outer rivet at a distance of 41 inches from the neutral axis is allowed to take a stress of 4330 pounds (as explained above), the stress in the rivet at the distance of $y$ is $(4330y \div 41)$ pounds, and its moment about the neutral axis is $(4330y^2 \div 41)$ pound-inches. For all the rivets in one row, both above and below the neutral axis, the moment is $\frac{4330}{41}\Sigma y^2$. Substituting the squares of the above distances $y$ (Pocket Companion, page 250) and adding them, the following results are obtained for the two rows of rivets:

$$2 \times \frac{4330}{41} \times 7351.6 = 1\,552\,800 \text{ pound-inches.}$$

$$2 \times \frac{4330}{41} \times 7009.6 = \underline{1\,480\,600}$$

$$3\,033\,400$$

The specifications do not give a unit stress for the web in tension, but as the shear given is 6000 the tensile strength will be taken at 9000 pounds. The holes in the web splices are punched (¶ 111), while those in the flange are drilled (¶ 110), and hence the unit stress should be less than for the flange.

The resisting moment of the solid web is

$$9000 \times \tfrac{1}{6} \times \tfrac{3}{8} \times 87 \times 87 = 4\,257\,600 \text{ pound-inches,}$$

and the value of the net section will be most conveniently obtained by deducting the moment due to the rivet holes. The deduction for a rivet hole at the distance of the outer fibre from the neutral axis is $9000 \times 1 \times \frac{3}{8} = 3500$ pounds, and for the entire second row of rivet holes whose distances are given above the moment is

$$2 \times \frac{3500}{43.5} \times 7009.6 = 1\,128\,000 \text{ pound-inches,}$$

leaving the resisting moment of the net section 3 129 600 pound-inches. It is seen therefore that the two rows of rivets (27 in the first and 26 in the second) have a strength equal to 97 per cent of the strength of the net section of the web. If, however, the rivets at 36 inches from the neutral axis may take a stress of 4330 and the flange rivets be excluded the moment becomes 2 689 000 pound-inches, or 86 per cent of the web's strength. These values might be increased by reducing the pitch of the rivets to $2\frac{3}{4}$ inches, or even to $2\frac{5}{8}$ (¶ 47) if necessary. It may be of interest to notice that one sixth of the net area of the web considered as a part of the flange would give a moment of

$$\tfrac{1}{6} \times \tfrac{3}{8}(87 - 26)9000 \times 86.8 = 2\,978\,300 \text{ pound-inches,}$$

which differs but little from the previous determination.

For rivets bearing on a $\frac{3}{8}$-inch web the maximum shear at the support would require $183\,700 \div 4330 = 42.4$ rivets.

If then the arrangement of rivets given above be adopted in the present design, the flange rivets will not be appreciably overstrained on account of unequal strains in the flange angles and web at the centre of the girder, and probably not at any other section.

It only remains to see what number of rivet holes the web section will allow to be taken out at the 10-foot section. The net area required is 141 100 ÷ 6000 = 23.52; gross area 32.62; and difference 9.10 square inches; and rivets allowed 9.10 ÷ 0.375 = 24.3 (¶ 31 *l*, 36). As in the inner row of rivets for the web sheet 24 rivet holes are in the same line and the rivets in the flange stagger with these, the arrangement will barely pass.

## ART. 38. WEB STIFFENERS.

(See paragraphs 42, 78, 79, 80, 81, 83, 84 and 98 of the Specifications in Art. 30.)

At any point of the web there exist compressive and tensile stresses at right angles to each other, whose intensities are equal to that of the vertical and horizontal shear at that point. The lines of maximum compressive and tensile stress cross each other at right angles at the neutral surface and make angles of 45 degrees with that surface. The compressive stresses tend to buckle the web plate, while the tensile stresses aid the web to resist buckling. It seems to be impossible to determine theoretically what size angles are needed as stiffeners; various assumptions are accordingly made in order to get some indication of what may be considered safe. The specifications state in ¶ 79 at what depth of girder stiffeners are regarded as necessary.

As the distance from the front of the vertical leg of a flange angle to the edge of its horizontal leg is $5\frac{1}{4}$ inches, let $5 \times 3\frac{1}{2}$ inch angles be adopted as stiffeners. The dimension of $3\frac{1}{2}$ inches was referred to in the preceding Article in considering stiffeners as related to the width of the splice plates. Where there are no splice plates, two $3\frac{1}{2} \times \frac{3}{8}$ inch fillers will be placed under each stiffener in order to avoid bending the stiffeners over the flange angles (¶ 81). Where there are splice plates or re-inforcing webs only half as many fillers are required.

If the angles and fillers be regarded as a column with the shear as a load it is found that a thickness of $\frac{9}{16}$ inch (¶ 111) for the angles would enable them to sustain over 75 per cent of the shear at the 5-foot section, and that a thickness of $\frac{5}{8}$ inch would enable them to support the entire shear at the 20-foot section. At the 5-foot section the re-enforcing plates afford greater stiffness than if only a filler $3\frac{1}{2}$ inches wide were used. The same is true in a lesser degree at the web splices. While these computations do not completely solve the problem it may be safely assumed that these stiffeners afford ample protection against buckling at the sections named. The following thicknesses of angles will then be chosen: At 5'. $\frac{9}{16}''$; at 10', $\frac{1}{2}''$; at 15', $\frac{7}{16}''$; at 20', 25', 30', 35' and 40', $\frac{3}{8}''$. The column formula employed in the computations leading to the results stated above was that given in the specifications for the intermediate posts of trusses, viz. $9000\left(1 + \frac{\text{min.}}{\text{max.}}\right) - 48\frac{l}{r}$, which gave a unit stress of 8740 pounds for the angles one-half inch thick and their fillers. The radius of gyration was 2.38, varying about 0.07 for each change of $\frac{1}{16}$ inch in the thickness of the angles (Pocket Companion, page 103).

Making a similar computation for the end of the girder, using the reaction as a load, but disregarding the fillers, it is found that six angles $\frac{7}{16}$ inch thick would give the necessary area. If there were no re-inforcing webs two would be placed at the outer end of the sole plate and four at the inner end, since the deflection of the girder causes the pressure on the inner end of the support to be greater than that at the other. In view of the fact that there are two re-inforcing web plates only four angles will be used, two at each end of the bearing plate (¶ 80), their thickness being $\frac{1}{2}$ inch. The main web as well as the re-inforcing webs should be finished so as to take bearing.

If the entire reaction were transmitted from the angles to the web through the connecting rivets it would require $183\,700 \div 7940 = 23.1$ rivets, the double shear being less than the bearing in three webs of $\frac{3}{8}$ inch each. As the pitch of the rivets in the stiffeners at the splices has been fixed at 3 inches the same pitch will be used throughout. This facilitates shop work and gives a sufficient number of rivets for all the stiffeners.

The end cover plate (¶ 98) will be attached by two rows of rivets to each end angle, the pitch in each row being 6 inches and the rivets staggered in adjacent rows. The rivets also stagger with those uniting the stiffeners to the web of the girder.

## ART. 39. RIVET SPACING IN FLANGES.

(See paragraphs 83 and 86 of the Specifications in Art. 30.)

The rivets uniting the web to the upper flange of a girder between two given sections have two duties to perform: first, to transfer whatever load rests on this division from the flanges, which directly receive it, to the web; second, to transfer from the web to the flanges the increment of flange stress between these sections. The required number of rivets must then be such as to safely transfer these stresses when their resultant is a maximum. The second factor in this maximum is considerably greater than the first except at the middle of the girder, and there the maximum allowable pitch usually requires a larger number of rivets than is theoretically needed. The maximum difference of flange stress in any two sections occurs when their difference of bending moment is a maximum, provided the effective depth remains the same. The variation in the effective depth, due to the added cover plates, is usually very small, being about two per cent in the girder under consideration. If the sections are at a distance apart equal to $dx$ the difference of moments ($dM$) is a maximum when the load is so placed as to cause the vertical shear ($V$) to be a maximum, since from mechanics $dM = Vdx$. When the distance between the sections is greater than $dx$ the difference of moments is a maximum when the loads are so placed that the vertical shear is a maximum in the section nearer the middle of the span, and this holds true for uniform loads until the sections are separated a distance somewhat less than the half span. As the sections are not taken farther apart than the depth, which even in short spans is generally less than one eighth of the span, the exact value of the limiting distance referred to need not be determined.

As it is customary to make the rivet spacing the same in the upper and lower flanges to facilitate shop work, the spacing is determined by the flange requiring the larger number of rivets. (See Chap. XI.)

When the lateral bracing is directly attached to the flange of a deck girder the greatest stress in the flange rivets occurs in the upper flange of the leeward girder, but when the lateral bracing is attached to the web adjacent to the flanges, then the greatest number of rivets required may be theoretically in any flange except the lower flange of the windward girder, depending upon the relative values of the flange stresses of the upper lateral system to those produced by the overturning moment of the wind. Both of these conditions will now be considered.

In the first case the flange stress of the lateral system does not need to be taken into account. The simultaneous moments in pairs of sections when the live load is placed as indicated above, as well as their differences, are given in Art. 31. The moments are laid off as ordinates in Fig. 29, each pair being joined by a straight line; their differences are laid off in Fig. 30 as ordinates above the axis $AB$ at the middle of each division of the girder. By considering the overturning effect of the wind on the train as a uniform load covering the entire girder the bending moments due to all loads other than the live load will form the ordinates to a parabola whose vertex is at $E''$ in Fig. 29 and whose ordinate $BE''$ at the middle of the girder is equal to 624 000 + 548 600 + 121 400 = 1 294 000 pound-feet (Art. 34). If $CD'$ in Fig. 30 be laid off equal to the ordinate at the 5-foot section in Fig. 29, and a right line be drawn from $D'$ to $B$, the middle of the girder, the ordinates

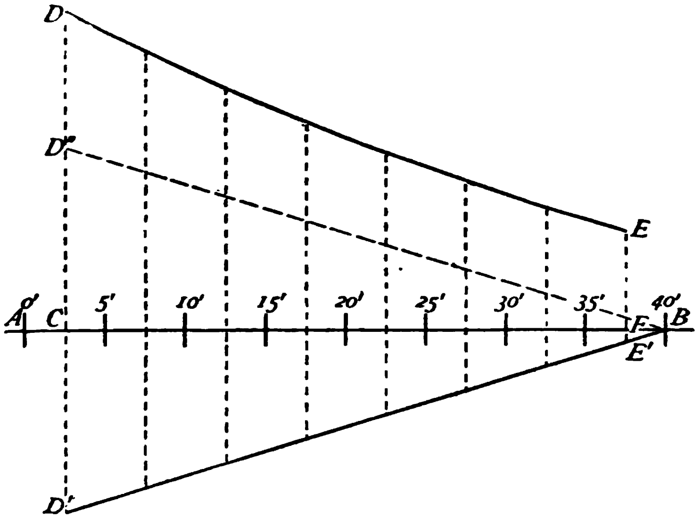



FIG. 30.

to this line will equal the differences between the adjacent ordinates (5 feet apart) of the parabola. The value of $CD'$ is readily computed to be $1\,294\,000 - 1\,294\,000(\frac{7}{8})^2 = 303\,000$ pounds. If $CD''$ be laid off equal to $CD'$ it is seen that $D''B$ is almost parallel to $DE$. This shows that the error involved in assuming the effect of the wind on the train, as a uniform load covering the entire girder, while the train really occupies different positions for maximum shear in the successive sections, is comparatively very small.

The total differences of moments in the pairs of sections are given in the table opposite.

In order to reduce the differences of moments to differences of flange stresses it is necessary to divide them by the effective depth in each division, but this depends on the number of

| Division of Girder. | Difference of Moments. | Effective Depth. | Difference of Flange Stresses. | No. of Rivets for Hor. Comp. of Stress. | No. of Rivets for Vert. Comp. of Stress. | Total Number of Rivets. |
|---|---|---|---|---|---|---|
| | Pound-feet. | Inches. | Pounds. | | | |
| 0′–5′ | 833 000 | 85.1 | 117 460 | 27.2 | 2.9 | 30.1 |
| 5′–10′ | 728 000 | 85.1 | 102 660 | 23.7 | 2.9 | 26.6 |
| 10′–15′ | 628 000 | 85.1 | 88 560 | 20.5 | 2.9 | 23.4 |
| 15′–20′ | 527 000 | 86.0 | 73 530 | 17.0 | 2.9 | 19.9 |
| 20′–25′ | 436 000 | 86.8 | 60 280 | 13.9 | 2.9 | 16.8 |
| 25′–30′ | 351 000 | 86.8 | 48 530 | 11.2 | 2.9 | 14.1 |
| 30′–35′ | 271 000 | 86.8 | 37 470 | 8.7 | 2.9 | 11.6 |
| 35′–40′ | 185 000 | 86.8 | 25 580 | 5.9 | 2.9 | 8.8 |

cover plates, and hence the length of the cover plates must be determined at least approximately. The maximum ordinate in Fig. 29 corresponds to an area of 37.71 square inches (end of Art. 35). If the pitch were 6 inches the area for the angles and the first cover plate would be 22.57, and for the angles and two cover plates 30.39 square inches, as given in Art. 35. By a simple graphical operation the lengths of the ordinates in Fig. 29 corresponding to these areas are readily obtained. The position of the shorter ordinate is found to be a little on the left of the 15-foot section, but as the pitch there will be less than 6 inches the second cover plate will probably extend almost to the 10-foot section. The longer ordinate is found to be on the right of, and near to, the 20-foot section. The first cover plate is to extend to the end (¶ 90). The effective depths are 85.0, 86.0 and 86.8 inches for the one, two and three cover plates respectively when the pitch is taken as 6 inches, but as the pitch toward the end of the girder will be less than 6 inches the depth of 85.0 is replaced by 85.1 inches. The third column of the table shows the depths used.

After dividing the differences of flange stresses by 4330 pounds (the strength of one rivet in bearing on the ⅜-inch web (¶ 31*m*)) the quantities in the succeeding column are obtained.

Now when the live load is in the position producing the maximum difference of flange stress in a division the first driver is at its right-hand end, the pilot being in the preceding division (Art. 31). Assume one half of the driver as being on the division. The weight of 5 feet of track for one girder is $5 \times 200 =$ 1000 pounds. The transfer of live load from the windward to the leeward rail by the overturning effect of the wind on the train for a five-foot division is $300 \times 5 \times 7 \div 7 = 1500$ pounds. The total load is then $10\,000 + 1000 + 1500 = 12\,500$ pounds. If all of this load or the larger part of it were distributed over the division the proper method of finding the maximum stress on the rivets would be to find the resultant whose horizontal component is the increment of flange stress and whose vertical component is this load, but as most of this load is concentrated on a very narrow limit at one end of the division the number of rivets due to each component will be added. The number of rivets for the vertical component is $12\,500 \div 4330 =$ 2.9. The total number of rivets in each division is given in the last column of the table.

In the second case where the lateral bracing is attached to the web alongside of the flange the flange stresses of the lateral systems must be included in the computation. The equivalent bending moment of the upper lateral system when reduced to the same depth as the girder is $348\,000 \times 86.8 \div 84$ $= 359\,600$ pound-feet, and that for the lower system is $92\,000$ $\times 86.8 \div 84 = 95\,100$. The moment at the center, due to all loads but the live load, for the upper flange of the leeward girder is $624\,000 + 548\,600 + 121\,400 - 359\,600 = 934\,400$ pound-feet, and for the lower flange of the leeward girder is $624\,000 + 548\,600 + 121\,400 + 95\,100 = 1\,389\,100$ pound-feet. The table on the next page shows the resulting stresses.

After dividing the differences of stresses by 4330 the numbers obtained for the upper flange are increased by 2.9 as

| | Upper Flange. | | Lower Flange. | |
|---|---|---|---|---|
| Division of Girder. | Difference of Moments. | Difference of Flange Stresses. | Difference of Moments. | Difference of Flange Stresses. |
| 0'–5' | 749 000 | 105 620 | 855 600 | 120 650 |
| 5'–10' | 654 800 | 92 330 | 747 200 | 105 360 |
| 10'–15' | 565 600 | 79 760 | 643 800 | 90 780 |
| 15'–20' | 476 400 | 66 470 | 540 300 | 75 390 |
| 20'–25' | 397 200 | 54 910 | 446 900 | 61 780 |
| 25'–30' | 323 000 | 44 650 | 358 500 | 49 560 |
| 30'–35' | 253 800 | 35 090 | 275 100 | 38 030 |
| 35'–40' | 179 600 | 24 830 | 186 700 | 25 810 |

before, while those for the lower flange are left unchanged. The results, together with those in the first case, are inserted in the following table:

| Division of Girder. | Case 1. Upper Flange. | Case 2. Upper Flange. | Case 2. Lower Flange. | No. of Rivets required by ¶ 83. | Number used in Design. |
|---|---|---|---|---|---|
| 0'–5' | 30.1 | 27.3 | 27.9 | 29.3 | 28 + 5 |
| 5'–10' | 26.6 | 24.2 | 24.3 | 25.8 | 29 |
| 10'–15' | 23.4 | 21.3 | 21.0 | 22.5 | 29 |
| 15'–20' | 19.9 | 18.3 | 17.4 | 19.2 | 20 |
| 20'–25' | 16.8 | 15.6 | 14.3 | 16.1 | 20 |
| 25'–30' | 14.1 | 13.2 | 11.5 | 13.3 | 20 |
| 30'–35' | 11.6 | 11.0 | 8.8 | 10.6 | 20 |
| 35'–40' | 8.8 | 8.6 | 6.0 | 7.9 | 20 |

In both cases the number of rivets shown in the table is not the real maximum for one or perhaps two divisions near the middle of the girder, since the maximum load on a division is 20 000 + 1000 + 1500 = 22 500 pounds, which alone requires 5.2 rivets. Under this load the difference of flange stresses is considerably reduced.

The fifth column of the above table gives the number of flange rivets required in each division in accordance with ¶ 83 of the specifications, the depth employed being the depth of the web, or 87 inches. If, instead of this depth, 82 inches or the distance between the outer row of flange rivets were employed the results would be 31.1. 27.4, 23.9, 20.4, 17.1, 14.1, 11.2 and 8.4. The last column shows the number actually used. A rivet pitch of 4 inches gives 28 rivets in the first division and 5 rivets on the left of the o′ section which are really available for this division. As it is desirable to have but few changes in pitch the 4-inch pitch is continued to the 15-foot section, then changed to 6 inches, and this continued to the middle. The requirement of ¶ 86 has already been alluded to in Art. 35 as referring to successive flange rivets, whether they are in the same or in adjacent rows.

The theoretic number of rivets required to connect the flange angles with the cover plates may be obtained by computing the horizontal shear between these parts of the flange. It is seen therefore that in passing from the inner to the outer cover plate a decreasing number of rivets is required. In order, however, that the net section in tension may not be reduced too much these rivets are staggered with those connecting the flange angles to the web. It is also cheaper usually to make a considerable increase in the number of rivets than to construct an additional pattern for the spacing.

### ART. 40. LENGTHS OF COVER PLATES.

(See paragraphs 90 and 91 of the Specifications in Art. 30.)

In the preceding Article the rivet pitch in the flanges was determined, the pitch being 4 inches from the end to the fifteen-foot section, and 6 inches from there to the middle of the girder.

According to ¶ 82 the net section for a pitch of 4 inches requires 3.55 rivet holes to be deducted from the gross section of each angle and 3.28 rivet holes from plates, for in Fig. 27, $e + f + e = 2.01 + 3.61 + 2.01 = 7.63$, and in Fig. 28, $2e = 2 \times 2.01 = 4.02$. The remainder of the computation is similar to that in Art. 35. Hence the net section is composed of

| | | | |
|---|---|---|---|
| 2 angles, | $6 \times 6 \times \frac{3}{8}$, | $2(8.44 - 2.66) =$ | 11.56 sq. in. |
| 1 plate, | $16 \times \frac{5}{8}$, | $(16 - 3.28)\frac{5}{8} =$ | 7.95 |
| | | | 19.51 |

The center of gravity of the net section from the horizontal backs of the angles is next obtained:

| | | |
|---|---|---|
| 2 angles, $6 \times 6 \times \frac{3}{8}$, | $16.88 \times 1.77$ | $= 29.88$ |
| Less 7.1 rivet holes, | $5.32\left(\frac{4.75 + 2.5 + 2 \times 0.375}{4}\right)$ | $= 10.64$ |
| | 11.56 | 19.24 |
| 1 plate $(16 - 3.28)\frac{5}{8}$ | $7.95 \times \frac{5}{16}$ | $= 2.48$ |
| | 19.51 sq. in. | 16.76 |

This distance is then $16.76 \div 19.51 = 0.86$ inches. The corresponding distance of the center of gravity of the gross section is found to be 1.00 inch. The effective depth is hence $87.0 - (1.0 + 0.9) = 85.1$ inches. The unit stress is 12 590 pounds (Art. 34).

The bending moment corresponding to the above net section is $19.51 \times 12\,590 \times 85.1 \div 12 = 1\,741\,900$ pound-feet. To find the location of this moment, the live-load bending moments given in Art. 31 are laid off as ordinates in Fig. 29, giving the curve $ACDE$ for the left half of the girder. The corresponding moment curve for all the other loads is the parabola $AC'D'E'$, $BE'$ being equal to 1 389 100 pound-feet. The ordinate which measures off 1 741 900 pound-feet is now found at the distance of 28.7 feet from the middle.

The net section of the angles and two cover plates is 30.39 sq. in., the effective depth 86.0 inches, and the corresponding bending moment 2 742 000 pound-feet, which ordinate is located at 18.45 feet from the middle. Each plate will be extended beyond its theoretical limit so as to receive four additional rivets (¶ 91), thus placing the end of the second cover plate at 29 feet 6½ inches and the third at 19 feet 7 inches from the center. The first plate (⅝ inch thick) extends to the end of the girder (¶ 90).

It has been customary to reduce the pitch of the rivets at the end of cover plates. If this were done in the case of the third plate it would reduce the net section of the entire flange at that point and require (¶ 82) the extension of the third plate considerably farther. No such reduction in pitch should be made and these specifications (¶ 91) indicate that the "regular pitch" shall be maintained.

## ART. 41. FLANGE SPLICES.

(See paragraphs 46, 93, 94 and 95 of the Specifications in Art. 30).

The angles and covers should be as long as practicable (¶ 46) to reduce splices to a minimum. The first cover plate will be spliced at the middle of the girder and the others made continuous. One angle will be spliced a few feet on the left, and the other on the right, of the middle (¶ 95).

The splice plate for the first cover plate will be an outside cover, which having to transmit the full strength of the ⅝-inch plate must have the same thickness. The stress is $(16 - 2.1)\frac{5}{8} \times 12\,590 = 109\,410$ pounds, and, since the rivets are weaker in single shear than in bearing for a plate of this thickness, it will require $109\,410 \div 3970 = 27.6$ shearing surfaces. If the cover and its splice plate were adjacent 28 rivets would answer, but as two other plates lie between them the rivets convey the stress from one to the other indirectly and hence some additional rivets should be allowed, say two for each intermediate plate, or 32 in all.

To splice the angles use a cover angle (¶ 94) on the side of the flange where each joint is located, and a vertical plate or bar as wide as the cover angle on the opposite side. The strength of one angle is $(8.44 - 1.50)12\,590 = 87\,370$ pounds and requires $87\,370 \div 3970 = 22.0$ shearing surfaces in the connecting rivets. If this number of rivets could be secured in each half of the cover angle without requiring a length exceeding the space between adjacent stiffeners this single cover would form the best splice, as the rivets would transmit the entire stress directly to it. The distance, however, between stiffeners is too small to allow this, and the pitch of the rivets cannot be reduced without weakening the entire flange by reducing the net section (¶ 82). Let the length of cover angle be chosen so as to include 16 rivets in each half, 8 being in the vertical and 8 in the horizontal leg of the angle. As there are two intermediate thicknesses of metal between the spliced angles and the vertical splice plate on the opposite side of the flange, not all of the 8 shearing surfaces may be counted. Since the top cover extends over this joint also, it may be used to give additional strength by prolonging it so as to add four more rivets. This will make the joint approximately of the same strength for each leg of the angles thus united. The thickness of the cover angle must be such that its strength in tension equals the strength of 16 rivets in single shear. This requires a net area of $16 \times 3970 \div 12\,590 = 5.04$ square inches. This area is furnished by a cover angle as wide as the inner-face width of the flange angles and $\frac{11}{16}$ inch thick. It would be better to make it $\frac{3}{4}$ inch.

The above computations all refer to the flange in tension. The maximum compressive flange stress is somewhat less than the tensile, hence the same dimensions and number of rivets will be used in the splices for both flanges (¶ 93). Economy of manufacture is promoted by this arrangement. (Chap XI).

## ART. 42. BED PLATES AND ROLLERS.

(See paragraphs 27, 28, 31 *o*, 43 and 155–160 of the Specifications in Art. 30.)

As the span exceeds 75 feet, a nest of turned-steel rollers is required at one end (¶ 158), the diameter of the rollers not being less than $2\frac{1}{2}$ inches. Assuming this diameter, the allowable pressure per linear inch of roller is $500 \times \sqrt{2.5} = 790$ pounds. The reaction of the support is 183 700, which would require $183\,700 \div 790 = 232.5$ inches of roller. Assuming 8 rollers and a clearance of half an inch between them, these would cover a space of about $20 \times 29\frac{1}{8}$ inches. The area of the bed plate must be (¶ 155) $183\,700 \div 250 = 734.8$ sq. in., or $24 \times 30\frac{5}{8}$ inches. This arrangement would pass, as the strength of the bed plate would permit projecting it beyond the rollers sufficient to give the requisite area on the masonry.

If the diameter be taken $3\frac{1}{2}$ inches the area covered by 7 rollers is $27\frac{1}{2} \times 28\frac{1}{8}$ inches, and by 6 rollers $23\frac{1}{2} \times 32\frac{3}{4}$ inches. The bearing surface should have its greater dimension at right angles to the girder in order that the pressure on the rollers may not be too unequally distributed by the deflection of the girder under full load. However, an increase in the length of the rollers requires a considerable increase proportionally in the strength of the bearing plates or sole plates which transfer the weight from the girder to the rollers. The dimensions $27\frac{1}{2}$ and $32\frac{3}{4}$ inches given above are for these reasons considered to be rather large. Let a diameter of 3 inches be tried. The allowable pressure per linear inch of rollers is 866 pounds, the aggregate length of rollers 212.1 inches, and 7 rollers cover an area of $24 \times 30\frac{1}{2}$ inches. Let this size be adopted.

The bearing plates in distributing the pressure on the rollers should not be strained beyond the safe limit. In order to compute the distance each plate may project beyond the one above it let the unit stress be assumed the same as in the chords. Remembering that the reaction under the projection is a uniform load of 866 pounds per linear inch, for each strip 3.5 inches wide the following quantities are obtained: The maximum moment $M = \frac{866x^2}{2}$ if $x$ is the projection in inches. The resisting moment of the strip of plate whose thickness is $d$ is $12\,590 \times 3.5 \times d^2 \div 6$. Equating these values and reducing, $x = 4.12\,d$.

As the stiffeners at each end of the bearing plate are fitted to the flange angles, which are $\frac{3}{4}$ inch thick, the load may be regarded as about uniformly distributed over the width of $12\frac{5}{8}$ inches. When $d$ is $\frac{5}{8}$ inch, $x$ becomes 2.57 inches, hence the projection of the cover plate of the girder of less than 2 inches is safe. If a bearing plate $\frac{3}{4}$ inch thick is placed below that it may project 3.09 inches, hence let it be made 22 inches wide. A plate 1 inch thick may project 4.12 inches, hence let a $1\frac{1}{16}$-inch plate be used, which may project far enough to cover the entire length of roller, or $30\frac{1}{2}$ inches.

The thickness of the bed plate will be taken at $\frac{13}{16}$ inch (¶¶ 43 and 155), being planed down (on one side) from a thickness of $\frac{7}{8}$ inch.

The greatest shear on the anchor bolts occurs when the heaviest live load is on the bridge and the brakes are applied when it is moving forward toward the roller end. This maximum live load is 215 000 pounds for each girder, the reaction at the forward end being 119 000 and at the other 96 000. The traction is $215\,000 \times 0.2 = 43\,000$ pounds (¶ 27). The dead-load reaction is 31 200, which added to 96 000 gives 127 200 pounds. The coefficient for steel on masonry is 0.25 (¶ 31 *o*), hence the frictional resistance of the fixed support is $127\,200 \times 0.25 = 31\,800$ pounds. This leaves a shear on the bolts of $43\,000 - 31\,800 = 11\,200$ pounds. If soft-steel anchor bolts are used at each support the area required is $11\,200 \div 6\,600 = 1.70$ square inches. Four bolts $\frac{3}{4}$ inch in diameter would supply this area. The diameter adopted is hence the minimum allowed (¶ 157) or $\frac{7}{8}$ inch.

The rollers are grooved at the middle and engage with a bar riveted to the bearing plate as well as with one riveted to the bed plate so as to prevent lateral motion of the girder. These bars may be $1\frac{3}{4} \times \frac{5}{16}$ inch in section, each being held in place by six or seven $\frac{5}{8}$-inch countersunk rivets. The anchor bolts at this end pass up through the bearing plate, in which slotted oval holes are cut of such a length as to allow a longitudinal movement of at least 0.8 inch (¶¶ 160 and 28).

At the fixed end a cast-iron base is used whose height equals that of bearing plates, rollers, and bed plate combined, and whose lower surface furnishes the necessary area on the masonry. Its form and dimensions are shown in Fig. 31.

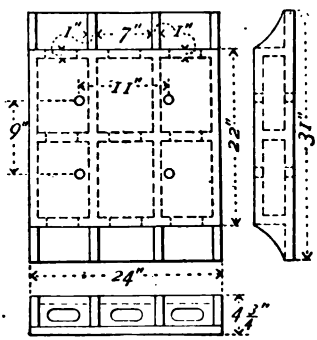


FIG. 31.

Since the deflection of a girder causes an unequal distribution of load on the bearings, some modification of their construction is necessary for girders of long span. During the last few years many plate girders have been built which exceed the limit mentioned in ¶ 3. Fig. 32 shows the end elevation, transverse section and side elevation of an expan-

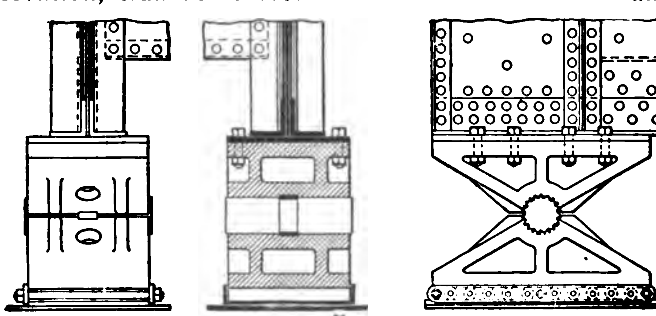


FIG. 32.

sion bearing designed by the Engineering Department of the New York Central and Hudson River Railroad for a plate girder having a span of 112 feet. The pin has a diameter of 6 inches, and is held in place by a ring one inch thick made in two sections. The thickness of metal in the cast pedestal is 2 inches. The rollers are 2 inches in diameter, have a bearing length of 20 inches, and are spaced $3\frac{1}{4}$ inches center to center except at the middle, where a tie-rod passes between the rollers.

## ART. 43. UPPER LATERAL SYSTEM.

(See paragraphs 31 *h*, *i*, 34, 37, 67, 87 and 89 of the Specifications in Art. 30.)

For the lateral systems the Warren type of truss will be employed, the center lines of the braces intersecting in the web of each girder at the five-foot divisions except at the ends, where the intersection will be one foot inside of the zero section which passes through the center of the bearing plate (¶¶ 87 and 89). The outline diagram of the upper system is shown in Fig. 33, and of the lower system in Fig. 34. The full

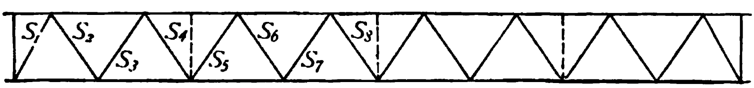



FIG. 33.

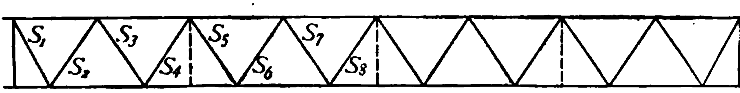



FIG. 34.

panel lengths are 10 feet, the width 7 feet, sec $\theta = 1.229$ and sec $\theta' = 1.152$. The panel load for that portion of the pressure of the wind on the girder and track which affects the upper system is $30 \times 10 \times 4.5 = 1350$ pounds, 4.5 feet being the average depth of the area exposed to the wind. Only the area of the windward girder is taken into account. The panel wind load for the moving train is $300 \times 10 = 3000$ pounds.

Assuming the moving wind load to be applied at the panel points on the leeward side, the following maximum stresses are obtained, the two values for each member being due to the wind blowing in opposite directions.

| $S_1$ | $S_2$ | $S_3$ | $S_4$ | $S_5$ | $S_6$ | $S_7$ | $S_8$ |
|---|---|---|---|---|---|---|---|
| − 18 320 | + 17 880 | − 14 660 | + 13 000 | − 10 240 | + 8570 | − 6270 | + 4610 |
| + 19 270 | − 17 100 | + 15 440 | − 12 440 | + 10 780 | − 8250 | + 6590 | − 4520 |

To obtain the sectional areas of the braces the specifications require the use of two formulas for unit stress (¶ 31 *h, i*):

For compression only, $12\,600 - 60\frac{l}{r}$;

For the greatest stress, $8\,400\left(1 - \frac{\text{max. lesser}}{2 \text{ max. greater}}\right)$.

The length of the diagonals measured between panel points is 8 feet $7\frac{1}{2}$ inches, but the braces will be somewhat shorter, hence 100 inches may be used as the length $l$. As it is desirable to use an angle with unequal legs on account of the stress due to the eccentric connection when only one leg of an angle is riveted to the connecting plate, let $4 \times 3$ inch angles be tried for those members consisting of only one angle.

The above formula for the greatest stress gives 4410 pounds per square inch for the unit stress in $S_1$, $S_3$, $S_5$ and $S_7$, and 4380 pounds for $S_2$, $S_4$, $S_6$ and $S_8$. The least radius of gyration $r$ for one $4 \times 3$ inch angle is 0.65 or 0.66, depending on the thickness (Pocket Companion, page 104). Using the smaller value, the formula for compression gives a unit stress of 3370 pounds per square inch. For two such angles placed back to back and riveted together on the 4-inch leg the mean value of $r$ is 1.24 (Pocket Companion, page 104 or 151), which gives a unit stress of 7760 pounds per square inch. This value being so much larger than the unit stress derived from the formula indicates that $3 \times 3$ inch angles may be used, and probably to advantage. For two $3 \times 3$ inch angles riveted together $r$ is 0.91 for a thickness of $\frac{3}{8}$ inch and 0.89 for a thickness of $\frac{9}{16}$ inch. Using $r = 0.90$, the formula for compression gives a unit stress of 5930 pounds per square inch. Assuming two angles to be needed for $S_1$, it will require a net area of $19\,270 \div 4410 = 4.37$ square inches. Two angles $3 \times 3 \times \frac{1}{2}$ inch give a net area of $2(2.75 - 0.44) = 4.62$ square inches (Pocket Companion, page 106), one $\frac{7}{8}$-inch hole being deducted from each angle (¶ 36), since $\frac{3}{4}$-inch rivets are to be employed for the entire lateral system (¶ 67).

It may be well perhaps to suggest to the student the desirability of using the tables in the handbook as far as possible to reduce the labor of computation. The deductions to be made for the rivet holes may be conveniently obtained from Pocket Companion, page 191, by entering the column of $1\frac{3}{4}$ inches which is $2 \times \frac{7}{8}$ inch and then dividing by two the areas given for the various thicknesses, as follows: For $\frac{3}{8}''$, 0.33; for $\frac{7}{16}''$, 0.38; for $\frac{1}{2}''$, 0.44; and for $\frac{9}{16}''$, 0.49 square inch.

As the unit stress for tension is considerably less than that for compression, and the total stresses nearly the same, $S_2$, $S_3$ and $S_4$ have their sections determined by the net area for tension. The net area required for $S_4$ is $13\,000 \div 4380 = 2.97$ square inches, which is less than the net area of one $4 \times 3 \times \frac{9}{16}$ inch angle. Testing this for compression it is found to require $12\,440 \div 3370 = 3.69$ square inches, which would require an angle $\frac{5}{8}$ inch thick. On account of ¶ 111 the thickness of $\frac{9}{16}$ inch will not be exceeded. $S_5$ must have a net section of $10\,780 \div 4410 = 2.44$ square inches and a gross section of $10\,240 \div 3370 = 3.04$ square inches. One $4 \times 3 \times \frac{1}{2}$ inch angle gives a net area of 2.81 and a gross area of 3.25 square inches. $S_6$ requires net and gross areas of 1.96 and 2.45 square inches, which are furnished by a $4 \times 3 \times \frac{3}{8}$ inch angle with corresponding sectional areas of 2.15 and 2.48 square inches. This being the minimum thickness it will also be chosen for $S_7$ and $S_8$. The composition of the upper lateral braces is then as follows:

| | | | |
|---|---|---|---|
| $S_1$, | 2 angles, $3 \times 3 \times \frac{1}{2}$ in. | $S_5$, | 1 angle, $4 \times 3 \times \frac{1}{2}$ in. |
| $S_2$, | 2 angles, $3 \times 3 \times \frac{1}{2}$ in. | $S_6$, | 1 angle, $4 \times 3 \times \frac{3}{8}$ in. |
| $S_3$, | 2 angles, $3 \times 3 \times \frac{3}{8}$ in. | $S_7$, | 1 angle, $4 \times 3 \times \frac{3}{8}$ in. |
| $S_4$, | 2 angles, $3 \times 3 \times \frac{3}{8}$ in. | $S_8$, | 1 angle, $4 \times 3 \times \frac{3}{8}$ in. |

Since three braces consist of one angle of the minimum thickness allowed it would have been a loss of material if a larger size than $4 \times 3$ inches had been selected. Were a single $4 \times 3$ inch angle substituted in the case of $S_3$ its gross area would be larger than that of both angles as given above, and the same holds true for $S_1$ and $S_2$.

The strength of a field rivet of soft steel, $\frac{3}{4}$ inch in diameter, in single shear is $0.4418 \times 5200 = 2300$ pounds (¶ 31 *l*), and in bearing against a $\frac{3}{8}$-inch plate is $\frac{3}{4} \times \frac{3}{8} \times 13\,200 = 3710$ pounds, and against a $\frac{1}{2}$-inch plate $\frac{3}{4} \times \frac{1}{2} \times 13\,200 = 4950$ pounds. By using connecting plates $\frac{1}{2}$ inch thick the strength of the rivet will be as strong in bearing as in double shear. In $S_1$, $S_2$, $S_3$ and $S_4$ the rivets are in double shear and require at each connection 5, 4, 4 and 3 rivets respectively. $S_5$, $S_6$, $S_7$ and $S_8$ require 5, 4, 3 and 3 shearing surfaces respectively.

The connecting plates are attached to the web of the girder immediately below the flange angles by means of four short angles, $5 \times 3\frac{1}{2} \times \frac{7}{16}$ inch. To find the number of rivets in one of these connections it is necessary to find the components, parallel to the girder, of the stresses in the braces which meet there, and to divide their sum by 4510 pounds, the bearing value of $\frac{3}{4}$-inch shop rivets in a $\frac{1}{2}$-inch plate and which is less than the double shear. Beginning at the end of the girder and passing toward the middle (alternately in each girder) the numbers of rivets theoretically required in the joints are 3, 5, 5, 4, 3, 3, 2, 2, 2. The numbers of field rivets joining the connecting angles to the web are respectively 4, 7, 7, 6, 5, 4, 3, 3, 3, the rivets being $\frac{7}{8}$ inch in diameter and in single shear. In both cases larger numbers are inserted than theory calls for, as no angle should have less than two rivets and the maximum allowable pitch (¶ 47) is not to be exceeded. It would be better not to put less than three rivets in each connection of a lateral brace.

The simplest arrangement and that which is generally followed for shorter spans is to attach these connecting plates directly to the flanges of the girder, the necessary connecting rivets being omitted when the flanges are riveted up in the shop. The specifications (¶ 69) state that "whenever the grip length of rivet exceeds $2\frac{1}{2}$ inches power-driven rivets will be insisted upon." In this case the grip length at the middle would far exceed this limit, as the flange is $3\frac{1}{16}$ inches thick. This item in the specifications is evidently introduced in order to exclude hand riveting in the field under such conditions, and it is assumed in this design that power riveting in the field is not available.

The arrangement adopted transfers the increments in the chord stresses of the upper lateral system to the web near the flange instead of directly into the flange. In the leeward girder this will simply reduce the stress to be transferred by the rivets from the web to the flange. In the windward girder the flange rivets usually have an excess of strength and can readily transfer all of it if necessary to the flange. In every design the method of attachment of the lateral system should be decided upon before the rivet spacing in the flanges is fixed so as to be sure to have ample strength. In Art. 39 the number of flange rivets in each division was found for both conditions. Another effect of the lateral braces not being attached directly to the flange is that the difference between the components of the stresses in each pair of braces normal to the girder tends to bend the web. This force varies from 1350 to 2810 pounds, but as the connection with the web is made adjacent to both stiffener and flange angle, and on both sides of the former, the bending moment is comparatively small. In a number of instances this component is transferred directly into the stiffener by means of a short angle, the stiffener having an excess of strength at that point.

For the braces consisting of a single angle when treated as a column the neutral axis for least radius of gyration makes an angle of nearly 45 degrees with its neutral axis when treated as a beam carrying its own weight (¶ 34), and as the moment of inertia about the latter axis is nearly four times as great as that with reference to the former the unit stress due to the weight of these braces need not be considered.

In practice it has not been the general custom to determine the unit stress due to the bending moment caused by the center line of the connections not coinciding with the axis of the member.

In Art. 17 an expression was deduced for the flexural unit stress in a tension member of rectangular section. If $M$ be the bending moment in a member due to its own weight or to the eccentric application of the direct tension or compression in the member, whose ends are regarded as hinged, the maximum unit stress due to this moment is given by J. B. JOHNSON'S formula

$$S = \frac{Mc}{I \pm \frac{Pl^2}{10E}}$$

in which $c$ is the distance to the neutral surface from the outer fiber in which the stress $S$ occurs, $I$ the moment of inertia of the cross-section of the member, $P$ its direct stress, $l$ its length, and $E$ the coefficient of elasticity of the material. The plus sign in the denominator is to be used when the direct stress $P$ is tension, and the minus sign when $P$ is compression.

In order to illustrate the effect of eccentric connections the unit stresses due to this cause and to the simultaneous action of the weight of the member will be computed for the brace $S_7$, which is connected by only one leg of the angle.

For flexure in a vertical plane the moment due to eccentricity in $S_7$ when it is subject to its maximum tension and the 4-inch leg of the angle is vertical is $6590(1.28 - 0.19) = 7185$ pound-inches, while that due to its own weight is 885 pound-inches when the span is 100 inches as previously assumed. Taking the value of $E$ as 28 000 000 the unit stress due to both moments is found by the above formula to be 2513 pounds tension. When $S_7$ is subject to its maximum compression the unit stress due to the same causes combined is 2086 pounds compression. When the 3-inch leg of the angle is placed vertically the corresponding unit stresses are 1756 and 1322 pounds. It is seen therefore that the unit stress is considerably less when the smaller leg is placed vertically.

If the above unit tensile stress be added to the unit stress due to the direct tension without eccentricity the sum exceeds by nearly 10 per cent the value allowed by the second formula in ¶ 31 *i*. As, however, its maximum flexure due to eccentricity occurs in a plane nearly at right angles to that in which it would bend as a column it may be considered sufficiently strong. This example indicates the importance of avoiding eccentric connections if possible.

The braces consisting of two angles are united by three intermediate rivets, washers of the same thickness as the connecting plates being used to keep the angles at a uniform distance apart. In this way the angles mutually support each other against flexure vertically. (See Art. 138 for an illustration of the great value of a stiff lateral system in case of an accident).

## ART. 44. LOWER LATERAL SYSTEM.

(The references to the Specifications are the same as in Art. 43.)

The wind panel load on the lower system is $30 \times 10 \times 46 \div 12 = 1150$ pounds, the average depth of the lower half of the girder being about 46 inches.

The greatest stresses in $S_2$ (see Fig. 34) are $+4240$ and $-4950$ for the wind blowing in opposite directions. For one $3 \times 3$ inch angle $r = 0.59$, and the unit stress for compression only is $12\,600 - 60\dfrac{100}{0.59} = 2430$ pounds. The other formula gives 4800 pounds per square inch. The gross area is $4950 \div 2430 = 2.04$ square inches. The area of an angle $3 \times 3 \times \frac{3}{8}$ inches is 2.11 square inches. The brace $S_1$ on account of being shorter requires a smaller section. In view, however, of the stress due to its own weight and the secondary stress due to eccentric connections let a $4 \times 3 \times \frac{7}{16}$ inch angle be selected for the first three braces from the end and a $4 \times 3 \times \frac{3}{8}$ inch angle for the rest. The stresses are so small that it is not necessary to compute them for more than two or three at the end.

The smallest compression member used in both lateral systems is composed of a single angle $4 \times 3 \times \frac{3}{8}$ inch. Its least width measures 2.50 inches, hence the greatest length allowed for such a member (¶ 37) is $45 \times 2.5 = 112.5$ inches. The longest lateral is somewhat less than 100 inches in length.

As regards wind stresses alone the lower lateral system would be unnecessary provided efficient transverse bracing were employed, the latter transferring the wind pressure on the lower part of the girder to the upper lateral system, whence it is carried to the supports. But the lateral systems not only provide for wind stresses, but also resist lateral vibration due to the live load, and experience shows that the lower lateral system performs an important function in this respect.

## ART. 45. TRANSVERSE BRACING.

(See paragraphs 96 and 97 of the Specifications in Art. 30.)

The upper lateral system is intended to convey the horizontal pressure of the wind on the train and the upper part of the girder to the ends of the girder whence it is to be carried to the supports by means of transverse bracing. The usual form of such a bracing or cross frame (¶ 97) is similar to the intermediate cross frame shown on Plate III. The horizontal reaction of the upper lateral system is 17 400 pounds, and if it be assumed that one half of the load passes to the support through each diagonal, the stress in the upper horizontal is $\pm$ 8700 pounds, and in the diagonals $8700 \times 1.305 = \pm 11\,340$ pounds, their inclination being about 40 degrees.

The smaller unit stress is derived from the formula in ¶ 31 *i*, which in this case gives 4200 pounds per square inch. The net section for the horizontal is $8700 \div 4200 = 2.08$ square inches, which requires a $4 \times 3 \times \frac{3}{8}$ inch angle. A thickness of $\frac{7}{16}$ inch would be better, in order to provide for the secondary stress. The same size would be taken for the lower horizontal although its stress is probably much less. The net area for the diagonals is $11\,340 \div 4200 = 2.70$ square inches, which is furnished by a $4 \times 3 \times \frac{1}{2}$ inch angle. The two diagonals cross each other back to back and are both riveted to a small connecting plate. Five rivets are required in the end of each diagonal and four at the end of the horizontal. Connecting plates $\frac{3}{8}$ inch thick complete the material needed for the end cross-frame. Another style of end cross-frame is shown on Plate III, which is simple in form, practically rigid, and which allows it to be extended down to the bottom of the stiffeners, thus avoiding any tendency to bend the stiffeners such as exists in the other form. No additional computation is required in its design. The web plate is made of the minimum thickness ($\frac{3}{8}$ inch), the horizontals being $4 \times 3 \times \frac{3}{8}$ inch angles, and the rivets near the corners are spaced 3 or $3\frac{1}{2}$ inches, the rest being spaced about 5 inches. The end lower lateral is attached to the web of the frame by means of a short angle.

The intermediate transverse bracing is inserted as an aid in securing general stiffness for the structure. The stresses due to the wind are transmitted through the lateral system and the girders. They will be located 20 feet apart and composed of $3 \times 3 \times \frac{3}{8}$ inch angles united by small plates $\frac{3}{8}$ inch thick.

## ART. 46. FINAL ESTIMATE OF WEIGHT.

The following weights are computed with the aid of the tables in the Pocket Companion, pp. 38, 40, 41, 197–202, 209, and 261–264.

*Material for One Half of the Girder.*

Flanges :

| | | | |
|---|---|---|---|
| 4 angles, $6'' \times 6'' \times \frac{3}{4}''$, 41′ 0″ long, | @ 28.7 lbs., | 4 707 pounds. | |
| 2 cover plates, $16'' \times \frac{5}{8}''$, 41′ 0″ long, | @ 34.0 | 2 788 | |
| 2 cover plates, $16'' \times \frac{9}{16}''$, 29′ $6\frac{1}{2}''$ long, } 2 cover plates, $16'' \times \frac{9}{16}''$, 19′ 7″ long, } | @ 30.6 | 3 007 | |
| | | 10 502 | |
| Less corners clipped from cover plates. . . . . . . . . . . | | 5 | 10 497 |

Web :

| | |
|---|---|
| 3 plates, $7' 3'' \times \frac{3}{8}''$, 10′ 0″ long, } 1 plate, $7' 3'' \times \frac{3}{8}''$, 11′ 0″ long, } 2 plates, $6' 3'' \times \frac{3}{8}''$, 7′ 2″ long, } 7 splice plates, $14'' \times \frac{3}{8}''$, 6′ 3″ long, } | 437.9 sq. ft. @ 15.3 lbs., 6 700 |

Stiffeners :

| | | | |
|---|---|---|---|
| 2 angles, $5'' \times 3\frac{1}{2}'' \times \frac{9}{16}''$, 7′ $1\frac{1}{2}''$ long, | @ 15.2 lbs., | 217 | |
| 6 angles, $5'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$, 7′ $1\frac{1}{2}''$ long, | @ 13.6 | 581 | |
| 2 angles, $5'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$, 7′ $1\frac{1}{2}''$ long, | @ 12.0 | 171 | |
| 9 angles, $5'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$, 7′ $1\frac{1}{2}''$ long, | @ 10.4 | 667 | |
| 25 fillers, $3\frac{1}{2}'' \times \frac{3}{8}''$, 7′ $1\frac{1}{2}''$ long, | @ 4.47 | 796 | |
| | | | 2 432 |

Flange Splices:

| | | | |
|---|---|---|---|
| 2 cover angles, 5½″ × 5½″ × ¾″, 4′ 0″ long, | @ 24.9 lbs., | 199 | |
| 2 flats, 5½″ × ¾″, 4′ 0″ long, | @ 13.4 | 107 | |
| 2 cover plates, 16″ × ⅝″, 4′ 7″ long, | @ 34.0 | 156 | |
| | | 462 | |
| Less corners clipped ... | | 6 | |
| | | | 456 |
| 1 end cover plate, 16″ × ⅜″, 7′ 3″ long,<br>1 corner cover plate, 16″ × ⅜″, 1′ 0″ long, | @ 20.4 lbs., | 168 | |
| Less corners clipped ... | | 3 | |
| | | | 165 |
| 1 bearing plate, 24″ × ¾″, 22″ long, | @ 61.2 lbs., | 112 | |
| 1 bearing plate, 24″ × 1 1/16″, 36″ long, | @ 86.[illegible] | 260 | |
| 1 flat, 1¾″ × 5/16″, 24″ long, | @ 1.86 | 4 | |
| | | | 376 |
| Total ... | | | 20 626 |

*One Half of Upper Lateral System.*

| | | |
|---|---|---|
| Braces: 2 angles, 3″ × 3″ × ½″, 6′ 8″ long,<br>4 angles, 3″ × 3″ × ½″, 7½″ long,<br>2 angles, 3″ × 3″ × ½″, 7′ 5″ long, | @ 9.4 lbs., | 288 pounds. |
| 2 angles, 3″ × 3″ × ⅜″, 7′ 7″ long,<br>2 angles, 3″ × 3″ × ⅜″, 7′ 5″ long, | @ 7.2 | 216 |
| 1 angle, 4″ × 3″ × ½″, 7′ 6″ long,<br>2 angles, 4″ × 3″ × ½″, 7½″ long, | @ 11.1 | 97 |
| 3 angles, 4″ × 3″ × ⅜″, 7′ 6″ long,<br>2 angles, 4″ × 3″ × ⅜″, 6″ long, | @ 8.5 | 200 |
| | | 801 |
| 8½ connecting plates, ½″ thick, aggregating 14.4 sq. ft. | @ 20.4 lbs., | 294 |
| Connecting angles: | | |
| 2 angles, 5″ × 3½″ × 7/16″, 12″ long,<br>8 angles, 5″ × 3½″ × 7/16″, 8½″ long,<br>8 angles, 5″ × 3½″ × 7/16″, 11″ long,<br>14 angles, 5″ × 3½″ × 7/16″, 7″ long, | @ 12.0 lbs., | 278 |
| 2 angles, 5″ × 3″ × ⅜″, 10″ long, | @ 9.8 | 16 |
| 16 washers, 3½″ diameter, ½″ thick, | | 16 |
| Total ... | | 1405 |

*One Half of Lower Lateral System.*

| | | | |
|---|---|---|---|
| Braces: | 1 angle, 4″ × 3″ × $\frac{7}{16}$″, 7′ 0″ long, | | |
| | 1 angle, 4″ × 3″ × $\frac{7}{16}$″, 7′ 5″ long, | @ 9.8 lbs., | 215 |
| | 1 angle, 4″ × 3″ × $\frac{7}{16}$″, 7′ 6″ long, | | |
| | 5 angles, 4″ × 3″ × $\frac{3}{8}$″, 7′ 6″ long, | @ 8.5 | 319 |
| | | | 534 |
| 8½ connecting plates, $\frac{3}{8}$″ thick, aggregating 10.9 sq. ft., | | @ 15.3 | 167 |
| Connecting angles: | | | |
| | 2 angles, 5″ × 3½″ × $\frac{3}{8}$″, 10″ long, | | |
| | 30 angles, 5″ × 3½″ × $\frac{3}{8}$″, 7″ long, | @ 10.4 lbs., | 212 |
| | 1 angle, 5″ × 3½″ × $\frac{3}{8}$″, 15″ long, | | |
| | 1 angle, 5″ × 3″ × $\frac{3}{8}$″, 10″ long, | @ 9.8 | 15 |
| | 1 angle, 5″ × 3″ × $\frac{3}{8}$″, 9″ long, | | |
| | Total | | 928 |

*End Cross Frame.*

| | | |
|---|---|---|
| 1 web plate, 6′ 6″ × $\frac{3}{8}$″, 6′ 3″ long, @ 99.45 lbs., | 622 | pounds. |
| 2 angles, 4″ × 3″ × $\frac{3}{8}$″, 6′ 6″ long, @ 8.5 | 110 | |
| Total | 732 | |

*Intermediate Cross Frame.*

| | | |
|---|---|---|
| 2 angles, 3″ × 3″ × $\frac{3}{8}$″, 6′ 6″ long, | | |
| 2 angles, 3″ × 3″ × $\frac{3}{8}$″, 7′ 9″ long, | @ 7.2 lbs., | 205 |
| 5 connecting plates, $\frac{3}{8}$″ thick, aggregating 3.3 sq. ft., | @ 15.3 | 51 |
| Total | | 256 |

*Dead Load for One Girder, excluding Track.*

| | | | |
|---|---|---|---|
| One plate girder, 2 × 20 626 pounds | | 41 252 | pounds. |
| One half upper lateral system | | 1 405 | |
| One half lower lateral system | | 928 | |
| One end cross frame | | 732 | |
| One and one half intermediate cross frames @ 256 lbs. | | 384 | |
| 3720 pairs of rivet heads in the girder, @ 0.444 lb., | 1652 | | |
| 416 pairs of rivet heads in one half of the lateral systems and cross frames, @ 0.268 | 111 | 1 763 | |
| Gross weight for a length of 82 feet | | 46 464 | pounds. |
| Net weight for a length of 80 feet (the span) | | 45 331 | pounds. |

The net weight assumed was 46 400 pounds, which is 1069 pounds in excess, or 2.3 per cent. As the shipping weights are generally allowed to vary 2.5 per cent, the weight assumed for the computation of stresses was as close as could be expected. It should be noted that ¶ 20 of the specifications is satisfied.

The weight of the end cross frame with open web is 385 pounds, and requires 53 rivets, and if this form were substituted for the one with the solid web the dead load for one girder would be reduced 702 pounds.

In order to show the relation which the weights of the various parts of this structure (exclusive of track) bear to each other and to the whole, the following table is presented.

| | Weight in pounds. | Percentage of total weight. | |
|---|---|---|---|
| Flanges | 20 994 | 45.2 | 74.0 |
| Web and its splices | 13 400 | 28.8 | |
| Stiffeners | 4 864 | 10.5 | 14.8 |
| Flange splices | 912 | 2.0 | |
| End covers | 330 | 0.7 | |
| Bearing plates | 752 | 1.6 | |
| | 41 252 | 88.8 | |
| ½ upper lateral system | 1 405 | 3.0 | 7.4 |
| ½ lower lateral system | 928 | 2.0 | |
| Transverse bracing | 1 116 | 2.4 | |
| Rivets | 1 763 | 3.8 | 3.8 |
| Total | 46464 | 100.0 | 100.0 |

This gives a better basis for a preliminary estimate of the dead load for one girder than any general formula that may be deduced. First assume a weight for one girder and half the lateral and transverse bracing of $6l$ to $7l$ per linear foot, the larger value being used for the heavier live loads, such as the one used in this design, and then compute the area of the flange sections and the thickness of the web in accordance with the given specifications. The weight of the girder might even be omitted entirely without materially changing the result, but it is better to include it in the manner just stated. The approximate length of cover plates can be readily obtained from the diagram of moments, and the number of splices in the web can be determined very quickly. Then let the weight of these items be increased by one third and the result will be found to differ but little from the final estimate.

In a stringer having a span less than 25 feet and under a much lighter live load the flanges and web may contain 78 or 80 per cent of the total weight. This arises from the fact that the web and flanges are continuous throughout, and stiffeners, if required at all, are comparatively light on account of the small depth.

## ART. 47. DISCUSSION OF THE ECONOMIC DEPTH.

It is of interest to observe what absolute as well as relative variations in the weight will be obtained by changing the depth of the web. Accordingly a revision of the above design was made for a depth of web of 96 inches. The number of rivets was, however, not recomputed, as it was assumed that the increase in rivets in the web would about equal the decrease in those required in the flanges. No increase was made in the sections of the stiffeners. The result is as follows:

| | Weight in pounds. | Percentage of total weight. | |
|---|---|---|---|
| Flanges | 18 918 | 40.8 | 72.7 |
| Web and its splices | 14 796 | 31.9 | |
| Stiffeners | 5 382 | 11.6 | 15.9 |
| Flange splices | 850 | 1.9 | |
| End cover | 362 | 0.8 | |
| Bearing plates | 752 | 1.6 | |

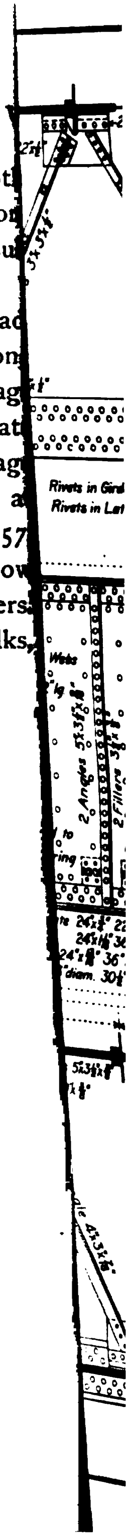
| | | | |
|---|---|---|---|
| One half upper lateral system .......... | 1 405 | 3.0 | 7.6 |
| One half lower lateral system........... | 928 | 2.0 | |
| Transverse bracing.................... | 1 202 | 2.6 | |
| Rivets ................................ | 1 763 | 3.8 | 3.8 |
| Total.......................... | 46 358 | 100.0 | 100.0 |

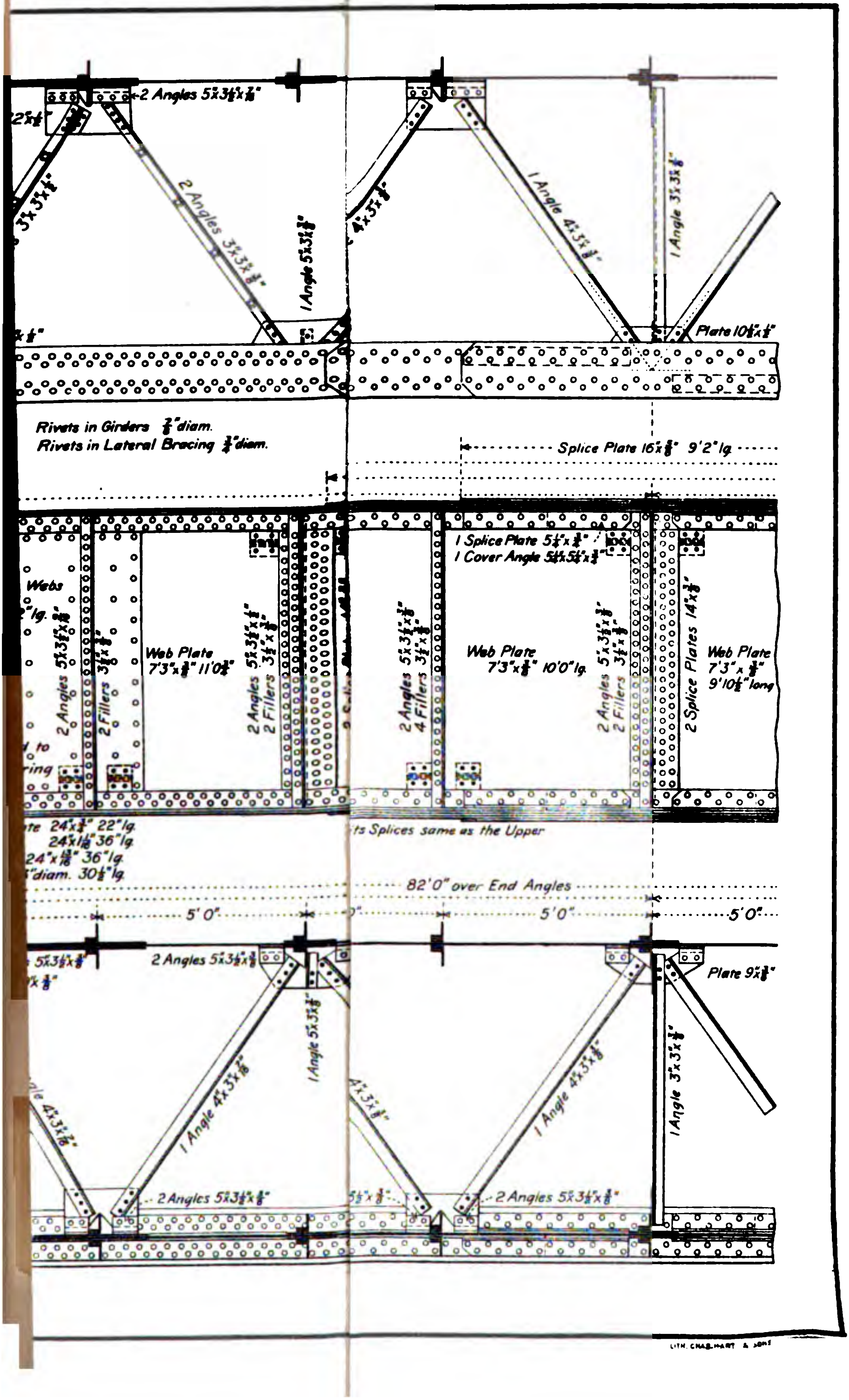
The reduction in the total weight is only 46 464 — 46 358 = 106 pounds. The weight in the flanges varies inversely as the effective depth, while that of the web and its details varies nearly as its own depth for relatively small changes in depth, and these two depths differ only by about one inch in this example. Slight changes from the economic depth do not appreciably affect the weight of the girder, hence these variations in depth should produce about equal changes in the weights of the flanges and of the web and its details. This occurs when their weights are about equal.

If the weights of the flanges be taken as inversely proportional to the depth of the web the weight for a depth of 96 inches is $(20\,994 + 912)87 \div 96 = 19\,852$ pounds, which is 84 pounds greater than that obtained by the revised design. The difference is due in part to the fact that in designing no changes less than $\frac{1}{16}$ inch were made in the thickness of angles and cover plates. The weight of the web and its details is $(13\,400 + 4864 + 330)96 \div 87 = 20\,518$ pounds, which is only 22 pounds less than the weight obtained by computing the increase in weight for the web, stiffeners, fillers, and end covers separately under the supposition that the slight increase in depth would not require any increase in section. For the depth of 87 inches the weight of the flange and its splices is $45.2 + 2.0 = 47.2$ per cent and that of the web and its details is $28.8 + 10.5 + 0.7 = 40.0$ per cent of the weight of the girder. For the depth of 96 inches the corresponding values are 42.7 and 44.3 per cent. For a depth of 94 inches the weights of these two items are almost exactly equal, and hence

on the basis of material alone this is the economic dept
The difference in cost of plates and angles and consideratio
relating to the cost of manufacture, however, modify this resu
to some extent. (See Art. 10.)

The student is referred to the following articles for ad-ditional examples of plate-girder construction: Two Lon Plate-girder Bridges, Engineering News, Vol. XXVII, pag 316, Apr. 2, 1892; Plate-girder Highway Bridge on Stat Street, Rockford, Ill., Engineering News, Vol. XXVII, pag 205, Feb. 27, 1892; and Plate-girder Highway Bridge a Brookline, Mass., Engineering News, Vol. XXVI, page 257 Sept. 19, 1891. Most of these are through bridges and sho different methods of connecting the floor system to the girders Two of the bridges have cantilever brackets for the sidewalks and the girders of one of them have curved upper flanges.




2 Angles 5"x3½"x⅞"
2 Angles 3"x3"x⅜"
1 Angle 5"x3"x⅜"
1 Angle 4"x3"x⅜"
1 Angle 3"x3"x⅜"
Plate 10½"x½"
Rivets in Girders ⅞" diam.
Rivets in Lateral Bracing ¾" diam.
Splice Plate 16"x⅝" 9'2" lg.
1 Splice Plate 5½"x½"
1 Cover Angle 5½"x5½"x½"
Web Plate 7'3"x⅜" 11'0½"
Web Plate 7'3"x⅜" 10'0" lg.
Web Plate 7'3"x⅜" 9'10½" long
2 Splice Plates 14"x⅜"
2 Angles
2 Fillers
4 Fillers
Webs
82'0" over End Angles
5'0"
2 Angles 5"x3½"x⅜"
Plate 9"x⅜"
1 Angle 5"x3"x⅜"
1 Angle 4"x3"x⅜"
1 Angle 3"x3"x⅜"
LITH. CHAS. HART & SONS

# CHAPTER VI.

## DESIGN OF A PIN TRUSS BRIDGE.

### ART. 48. DATA AND SPECIFICATIONS.

Let a single-track through railroad bridge be taken having single-intersection trusses of the Pratt type, and let the span be 142 feet between centers of end pins. The design is to be made in accordance with COOPER'S General Specifications for Iron and Steel Railroad Bridges and Viaducts, the required portions of which are reprinted below by permission of the author. The material to be used is wrought iron, excepting for the pins in the trusses. A clear width of 14 feet is required by ¶ 5 (of the Specifications), and as the cover plates for the end posts will probably be 20 or 22 inches wide, the distance center to center of trusses will be taken at 16 feet.

For this span the depth should be about one fifth of the span, or 28½ feet. On inspecting a few designs of Pratt trusses of somewhat different spans it is estimated that the required head room may be secured by a depth 2 feet less, and since it is proposed to include in the maximum stresses all the effects of the specified wind pressure the reduced depth of 26.5 feet between centers of chords will be adopted. The number of panels is preferably an odd number (Art. 9). Five panels would require a comparatively heavy floor system and would give poor proportions to the lateral systems, while seven panels would probably give the best results. However, in order to illustrate some points in the design of details six panels will be taken. The general drawing is shown on Plate IV and in Fig. 35 (Art. 49).

The following are the portions of COOPER'S Specifications which apply to this design:

GENERAL DESCRIPTION.

1. All parts of the structures shall be of wrought iron or steel, except ties and guard-rails. Cast iron or steel may be used in the machinery of movable bridges and in special cases for bed plates.

2. The following kinds of girders shall preferably be employed:

| | |
|---|---|
| Spans up to 16 feet.......... | Rolled beams. |
| Spans 16 to 70 feet.......... | Riveted plate girders. |
| Spans 70 to 100 feet.......... | Riveted plates or lattice girders. |
| Spans over 100 feet.......... | Pin-connected trusses. |

In calculating strains the length of span shall be understood to be the distance between centers of end pins for trusses, and between centers of bearing plates for all beams and girders.

3. The girders shall be spaced, with reference to the axis of the bridge, as required by local circumstances, and directed by the Engineer of the Railroad Company (¶ 5). Longitudinal floor girders shall in no case be less than three feet and three inches from the center line of tracks (¶ 6).

4. For all through bridges and overhead structures there shall be a clear head-room of 20 feet above the base of the rails.

5. In all through bridges the clear width from the center of the track to any part of the trusses shall not be less than seven (7) feet at a height exceeding one foot above the rails where the tracks are straight, and an equivalent clearance shall be provided where the tracks are curved.

10. Unless otherwise specified the form of bridge trusses may be selected by the bidder; but to secure uniformity in appearance it is desired that all "through" trusses shall be built with inclined end posts; for pin-connected trusses, preference shall be given to those of single intersections.

12. The wooden floors shall consist of transverse ties or floor timbers; their scantling will vary in accordance with the design of the supporting iron floor (¶ 15). They shall be spaced with openings not exceeding six inches, and shall be secured to the supporting girders by ¾-inch bolts at distances not over six feet apart. For deck bridges the ties will extend the full width of the bridge, and for through bridges at least every other tie shall extend the full width of bridge for a footwalk.

13. There shall be a guard timber (scantling not less than 6″ x 8″) on each side of each track, with its inner face parallel to and not less than 3 feet 3 inches from center of track. Guard timbers must be notched one inch over every floor timber, and be spliced over a floor timber with a half-and-half joint of 4 inches lap. Each guard timber shall be fastened to every third floor timber, and at each splice, with a three-quarter (¾)-inch bolt.

14. The guard and floor timbers must be continued over all piers and abutments.

15. The maximum strain allowed upon the extreme fiber of the best yellow-pine or white-oak floor timbers will be 800 pounds per square inch; the weight of a single engine wheel being assumed as distributed over three ties spaced as per ¶ 12.

16. The floor timbers from center to each end of span must be notched down over longitudinal girders so as to reduce the camber in the track, as directed by the Engineer.

LOADS.

23. All the structures shall be proportioned to carry the following loads: 1st. The weight of iron in the structure. 2d. A floor weighing 400 pounds per linear foot of *track*, to consist of rails, ties and guard timbers only. These two items, taken together, shall constitute the "dead load." 3d. For class A—A moving load for each *track*, supposed to be moving in either direction, and consisting of two "consolidation" engines coupled, followed by train weighing 3000 pounds per running foot. This "live load" being concentrated upon points distributed as in Diagram No. 3. Or, 80 000 pounds equally distributed upon two pairs of drivers, seven feet center to center; . . .

[Diagram No. 3 is not reproduced here, as the distances between the wheels are the same as those in Fig. 21, the loads, however, being 15 000 pounds on the pilot and tender wheels, 24 000 pounds on the drivers, and the uniform load 3000 pounds per linear foot.]

24. To provide for wind strains and vibrations, the top lateral bracing in deck bridges, and the bottom lateral bracing in through bridges, shall be proportioned to resist a lateral force of 450 pounds for each foot of the span; 300 pounds of this to be treated as a moving load. The bottom lateral bracing in deck bridges, and the top lateral bracing in through bridges, shall be proportioned to resist a lateral force of 150 pounds for each foot of the span. Preference will be given to lateral

bracing in the floor system, which is capable of resisting both compression and tension.

26. Longitudinally the bracing of the trestle towers and the attachments of the fixed ends of all trusses shall be capable of resisting the greatest tractive force of the engines, or any force induced by suddenly stopping upon any part of the work the assumed maximum trains; the coefficient of friction of the wheels upon the rails being assumed as 0.20.

27. Variations in temperature, to the extent of 150 degrees, shall be provided for.

29. All parts shall be so designed that the strains coming upon them can be accurately calculated.

## PROPORTION OF PARTS.

The following clauses are all intended to apply to wrought-iron construction.

30. All parts of the structure shall be proportioned in tension by the following allowed unit strains:

| | Pounds per square inch. |
|---|---|
| Floor-beam hangers, and other similar members liable to sudden loading (bar iron with forged ends) | 6 000 |
| Floor-beam hangers, and other similar members liable to sudden loading (plates or shapes), net section | 5 000 |
| Lateral bracing | 15 000 |
| Solid rolled beams, used as cross floor beams and stringers | 8 000 |
| Bottom flanges of riveted cross girders, net section | 8 000 |
| Bottom flanges of riveted longitudinal plate girders *over* 20 feet long, used as track stringers, net section | 8 000 |
| Bottom flanges of riveted longitudinal plate girders *under* 20 feet long, net section | 7 000 |

| | For live loads. | For dead loads. |
|---|---|---|
| Bottom chords, main diagonals, counters and long verticals (forged eyebars) | 8 000 | 16 000 |
| Bottom chords and flanges, main diagonals, counters and long verticals (plates or shapes), net section | 7 500 | 15 000 |

The areas obtained by dividing the live-load strains by the live-load unit strains will be added to the areas obtained by dividing the dead-load strains by the dead-load unit strains to determine the required sectional area of any member. (¶ 45.)

31. Angles subject to direct tension must be connected by both legs, or the section of one leg only will be considered as effective.

32. In members subject to tensile strains full allowance shall be made for reduction of section by rivet holes, screw threads, etc.

33. Compression members shall be proportioned by the following allowed unit strains:

| | | |
|---|---|---|
| Chord segments: | $P = 8000 - 30\frac{l}{r}$ | for live-load strains; |
| | $P = 16\,000 - 60\frac{l}{r}$ | for dead-load strains; |
| All posts: | $P = 7000 - 40\frac{l}{r}$ | for live-loads trains; |
| | $P = 14\,000 - 80\frac{l}{r}$ | for dead-load strains; |
| | $P = 10\,500 - 60\frac{l}{r}$ | for wind strains. |
| Lateral struts: | $P = 9000 - 50\frac{l}{r}$ | for assumed initial strain (¶ 34). |

$P =$ the allowed compression per square inch of cross-section.
$l =$ the length of compression member, in inches.
$r =$ the least radius of gyration of the section, in inches.

No compression member, however, shall have a length exceeding 45 times its least width.

34. The lateral struts shall be proportioned by the above formula (¶ 33) to resist only the resultant due to an assumed initial strain of 10 000 pounds per square inch upon all the rods attaching to them, assumed to be produced by adjusting the bridge or towers (¶ 41).

35. In beams and plate girders the compression flanges shall be made of same *gross* section as the tension flanges.

36. Riveted longitudinal girders shall have, preferably, a depth not less than $\frac{1}{10}$ of the span.

37. Members subject to alternate strains of tension and compression shall be proportioned to resist each kind of strain. Both of the strains shall, however, be considered as increased by an amountoequal to $\frac{8}{10}$ of the least of the two strains, for determining the sectional areas by the above allowed unit strains (¶¶ 30, 33).

38. The strains in the chords and end posts from the assumed wind forces need not be considered, except as follows: 1st, When the wind strains on any member exceed one quarter of the maximum strains due to the dead and live loads upon the same member. The section shall then be increased until the total strain per square inch will not exceed

by more than one quarter the maximum fixed for dead and live loads only. 2d. When the wind strain alone or in combination with a possible temperature strain can neutralize or reverse the tension in any part of the lower chord.

39. The rivets and bolts connecting the parts of any member must be so spaced that the shearing strain per square inch shall not exceed 7500 pounds, or $\frac{3}{4}$ of the allowed strain per square inch upon that member; nor the pressure upon the bearing surface per square inch of the projected semi-intrados (diameter × thickness of piece) of the rivet or bolt hole exceed 12 000 pounds, or one and a half times the allowed strain per square inch upon that member. In the case of field riveting the above limits of shearing strain and pressure shall be reduced one-third part. Rivets must not be used in direct tension.

40. Pins shall be so proportioned that the shearing strain shall not exceed 7500 pounds per square inch; nor the crushing strain upon the projected area of the semi-intrados of any member (other than forged eyebars, see ¶ 79) connected to the pin be greater per square inch than 12 000 pounds, or one and a half times the allowed strain per square inch; nor the bending strain exceed 15 000 pounds per square inch when the centers of bearings of the strained members are taken as the points of application of the strains.

41. In case any member is subjected to a bending strain from local loadings, such as distributed floors on deck bridges, in addition to the strain produced by its position as a member of the structure, it must be proportioned to resist the combined strains. If the fiber strain resulting from the weight only, of any member, exceeds ten per cent of the allowed unit strain on such member, such excess must be considered in proportioning the areas.

42. Plate girders shall be proportioned upon the supposition that the bending or chord strains are resisted entirely by the upper and lower flanges, and that the shearing or web strains are resisted entirely by the web plate; no part of the web plate shall be estimated as flange area. The distance between centers of gravity of the flange areas will be considered as the effective depth of all girders.

43. The webs of plate girders must be stiffened at intervals, about the depth of the girders, wherever the shearing strain per square inch exceeds the strain allowed by the following formula:

$$\text{Allowed shearing strain} = \frac{12000}{1 + \dfrac{H^2}{3000}},$$

where $H$ = ratio of depth of web to its thickness; but no web plates shall be less than three eighths of an inch in thickness.

45. The areas of counter rods shall be determined by taking the difference in areas due to the live- and dead-load strains considered separately (¶ 30); the counter rods in any one panel must have a combined sectional area of at least three square inches, or else must be capable of carrying all the counter live load in that panel.

## DETAILS OF CONSTRUCTION.

46. All the connections and details of the several parts of the structures shall be of such strength that, upon testing, ruptures shall occur in the body of the members rather than in any of their details or connections.

47. Preference will be had for such details as shall be most accessible for inspection, cleaning and painting; no closed sections will be allowed.

49. All web plates must have stiffeners over bearing points and at points of local concentrated loadings.

50. The pitch of rivets in all classes of work shall never exceed 6 inches, or sixteen times the thinnest outside plate, nor be less than three diameters of the rivet.

51. The rivets used shall generally be $\frac{3}{4}$ and $\frac{7}{8}$ inch diameter.

52. The distance between the edge of any piece and the center of a rivet hole must never be less than $1\frac{1}{4}$ inches, except for bars less than $2\frac{1}{2}$ inches wide; when practicable it shall be at least two diameters of the rivet.

57. Field riveting must be reduced to a minimum, or entirely avoided where possible.

58. The effective diameter of a driven rivet will be assumed the same as its diameter before driving. In deducting the rivet holes to obtain net sections in tension members, the diameter of the rivet hole will be assumed as $\frac{1}{8}$ inch larger than the undriven rivets.

59. When members are connected by bolts which transmit shearing strains, the holes must be reamed parallel and the bolts turned to a driving fit.

62. In compression members, abutting joints with planed faces must be sufficiently spliced to maintain the parts accurately in contact against all tendencies to displacement.

63. In compression members, abutting joints with untooled faces

must be fully spliced, as no reliance will be placed on such abutting joints. The abutting ends must, however, be dressed straight and true, so there will be no open joints.

65. Web plates of all girders must be arranged so as not to project beyond the faces of the flange angles, nor on the top be more than $\frac{1}{8}$ inch below the face of these angles, at any point.

67. In girders with flange plates, at least one half of the flange section shall be angles or else the largest-sized angles must be used.

69. The compression flanges of beams and girders shall be stayed against transverse crippling when their length is more than thirty times their width.

70. The unsupported width of plates subjected to compression shall not exceed thirty times their thickness; except cover plates of top chords and end posts, which will be limited to forty times their thickness.

71. The flange plates of all girders must be limited in width so as not to extend beyond the outer lines of rivets connecting them with the angles, more than five inches or more than eight times the thickness of the first plate. Where two or more plates are used on the flanges, they shall either be of equal thickness or shall decrease in thickness outward from the angles.

72. No iron shall be used less than $\frac{1}{4}$ inch thick, except for lining or filling vacant spaces.

73. The heads of eyebars shall be so proportioned and made that the bars will preferably break in the body of the original bar rather than at any part of the head or neck. The form of the head and the mode of manufacture shall be subject to the approval of the Engineer of the Railroad Company. The heads must be formed either by the process of upsetting and forging, or by the process of upsetting, piling and forging. No welding will be allowed in the body of the bars, nor in the process of piling, welding seams in any other direction than parallel to the sides of the original bars.

77. The lower chord shall be packed as narrow as possible.

78. The pins shall be turned straight and smooth, and shall fit the pin holes within $\frac{1}{50}$ of an inch, for pins less than $4\frac{1}{2}$ inches diameter; for pins of a larger diameter the clearance may be $\frac{1}{32}$ inch.

79. The diameter of the pin shall not be less than two thirds the largest dimension of any tension member attached to it. The several members attaching to the pin shall be so packed as to produce the least bending moment upon the pin, and all vacant spaces must be filled with wrought-iron filling rings.

85. Compression members shall be of wrought iron, and of approved forms.

86. The pitch of rivets at the ends of compression members shall not exceed four diameters of the rivets for a length equal to twice the width of the member.

87. The open sides of all compression members shall be stayed by batten plates at the ends, and diagonal latticework at intermediate points. The batten plates must be placed as near the ends as practicable, and shall have a length of $1\frac{1}{2}$ times the width of the member. The size and spacing of the lattice bars shall be duly proportioned to the size of the member. They must not be less than $2 \times \frac{1}{4}$ inch for posts 6 inches wide, nor $4 \times \frac{5}{8}$ inch for posts 15 inches wide. They shall be inclined at an angle not less than 60° to the axis of the member. The pitch of the latticing must not exceed the width of the channel plus nine inches.

88. Where necessary, pin holes shall be reinforced by plates, so the allowed pressure on the pins shall not be exceeded. These reinforcing plates must contain enough rivets to transfer their proportion of the bearing pressure, and at least one plate on each side shall extend not less than six inches beyond the edge of the batten plates. (¶ 87.)

89. Where the ends of compression members are forked to connect to the pins, the aggregate compressive strength of these forked ends must equal the compressive strength of the body of the members; in order to insure this result the aggregate sectional area of the forked ends, at any point between the inside edge of the pin hole and six inches beyond the edge of the batten plate, shall be about double that of the body of the member.

90. In compression-chord sections the material must mostly be concentrated at the sides, in the angles and vertical webs. Not more than one plate, and this not exceeding $\frac{1}{2}$ inch in thickness, shall be used as a cover plate, except when necessary to resist bending strains. (¶ 41.)

91. The sections of compression chords shall be connected at the abutting ends by splices sufficient to hold them truly in position.

96. In no case shall any lateral or diagonal rod have a less area than $\frac{3}{4}$ of a square inch.

97. The attachment of the lateral system to the chords shall be thoroughly efficient. If connected to suspended floor beams, the latter shall be stayed against all motion.

98. Preference will be given for a stiff angle-iron lateral system between the chords on the level of the floor.

99. All through bridges with top lateral bracing shall have wrought-

iron latticed portals, of approved design, at each end of the span, connected rigidly to the end posts. They shall be as deep as the specified head room will allow. (¶ 38.)

103. All bed plates must be of such dimensions that the greatest pressure upon the masonry shall not exceed 250 pounds per square inch.

104. All bridges over 75 feet span shall have at one end nests of turned friction rollers, formed of wrought iron or steel, running between planed surfaces. The rollers shall not be less than 2 inches diameter, and shall be so proportioned that the pressure per lineal inch of rollers shall not exceed the product of the square root of the diameter of the roller in inches multiplied by 500 pounds ($500\sqrt{d}$).

105. Bridges less than 75 feet span shall be secured at one end to the masonry, and the other end shall be free to move upon planed surfaces.

107. All the bed plates and bearings under fixed and movable ends must be fox-bolted to the masonry; for trusses, these bolts must not be less than 1¼ inches diameter; for plate and other girders, not less than ⅞ inch diameter. The contractor must furnish all bolts, drill all holes, and set bolts to place with sulphur.

108. While the roller ends of all trusses must be free to move longitudinally under changes of temperature, they shall be anchored against lifting or moving sideways.

109. All iron bridges with parallel chords shall be given a camber by making the panel lengths of the top chord longer than those of the bottom chord, in the proportion of ⅛ of an inch to every ten feet.

110. All bolts must be of neat lengths, and shall have a washer under the heads and nuts where in contact with wood.

Use of Steel.

118. Medium steel (¶ 139) may be used for tension members, plate girders, rolled beams and top-chord sections with an allowance of 20 per cent increase above allowed working strains on wrought iron; and for all posts by use of the following formulæ, in place of those given for wrought iron (¶ 33):

$$P = 8\,500 - 55\frac{l}{r} \quad \text{for live-load strains.}$$

$$P = 17\,000 - 110\frac{l}{r} \quad \text{for dead-load strains.}$$

$$P = 13\,000 - 85\frac{l}{r} \quad \text{for wind-load strains.}$$

Provided that, in addition to the previous details of construction, . . .

121. All rivets to be of steel.

123. Soft steel (¶ 141) may be used under the same conditions as wrought iron for all *riveted* work.

## ART. 49. FLOOR TIMBERS AND STRINGERS.

(See paragraphs 3, 12–16, 23, 24, 30–33, 35-37, 42, 43, 49, 58, 65, 67, 69, 71 and 110 of the Specifications in Art. 48.)

The iron floor system will consist of riveted stringers and floor beams, the latter being riveted to the posts of the trusses, just above the lower chord. The wooden cross-ties rest upon the stringers.

The alternate live load in ¶ 23, consisting of 80 000 pounds equally distributed on two pairs of drivers seven feet apart, will determine the section of the cross-ties for a given spacing of the stringers, which will be taken at 6 feet 6 inches between centers (¶ 3). The distance between centers of rails is 4 feet $10\frac{1}{2}$ inches, and the load on each rail for one tie is $20\,000 \div 3 = 6667$ pounds (¶ 15). The bending moment is $6667 \times 9.75 = 65\,000$ pound-inches, if the weight of rail, spikes and cross-ties be omitted as comparatively very small. For a unit stress of 800 pounds per square inch (¶ 15) the resisting moment equals $800bd^2 \div 6$. Assuming $b$ to be 8 inches and equating the moments, $d$ is found to be 7.8 inches. Ties $8 \times 8$ inches will therefore safely sustain all the loads, and they will be spaced 4 inches in the clear (¶ 12), which experience indicates as preferable to 6 inches, the maximum allowed. Every fourth tie will be bolted to the stringers (¶ 12), and the guard rails will be notched over the ties and bolted to them as specified in ¶ 13. The ties will be alternately 9 and 14 feet in length, a 2-inch plank walk two feet wide being laid on each side of the track. Track details not specified will be made the same as in the standard described in Art. 19.

As the design of a plate girder is explained in detail in Chap. V, and a stringer is a plate girder of short span and simple form, it will be necessary here to give only the results of the various steps in the computations and very brief, if any, descriptions of the methods employed.

The live-load stresses are as follows:

| Distance of Section from End. | Wheel at the Section. | Maximum Bending Moment. | Wheel at the Section. | Maximum Vertical Shear. |
|---|---|---|---|---|
| Feet. | | Pound-feet. | | Pounds. |
| 0 | .. | 0 | 5 | 33 940 |
| 3 | 5 | 84 000 | 5 | 27 580 |
| 6 | 5 | 130 500 | 5 | 21 520 |
| 9 | 4 | 167 500 | 5 | 14 830 |
| 10.58 | 4 | 171 750 | .. | .. |
| 11.83 | 4 | 169 500 | 5 | 9 590 |

These stresses were obtained from an equilibrium polygon including wheels 1 to 7 drawn on profile paper, Plate B, to scales of two feet to an inch and about 150 000 pound-feet to an inch. The maximum moments and shears are found in the right half of the stringer, as the first driver is farther from the second than the distance between the other drivers.

It may be of interest to the student to note the precision of the graphic method as compared with the analytic when the diagrams are drawn with reasonable care. By the analytic method if the distances on the tabulation diagram are expressed in hundredths of a foot the value obtained for the absolute maximum moment is 171 580 pound-feet, while if they are given only to the nearest tenth of a foot the result is 170 750 pound-feet. A high degree of precision in stresses is not required for the purpose of design, but it is important to know the relative precision of the graphic method, which saves so much time in obtaining the stresses.

The final estimate of the net weight of the plate girder in

Art. 46 gave a weight per linear foot of $7.1l$ pounds, $l$ being the span in feet. Considering the difference in live load, the absence of all splices in a stringer and the difference in unit stresses, let the weight of one stringer be assumed at $6.3l$ per foot. The weight of the stringer is then 3530 pounds, which added to one half the weight of the track for a panel length (¶ 23) makes the total dead load 8260 pounds. The live load transferred to the leeward stringer by the wind pressure on the train is $300 \times 23.67 \times 8 \div 6.5 = 8740$ pounds. As the stringer is riveted to the floor beam throughout the depth of the former, the lever arm of the wind pressure will be at least 8 feet. Theoretically it would be correct to take the lever arm as the distance from the center of pressure on the train to the mid depth of the stringer. The maximum moment due to the dead load and the load transferred by the wind is $(8260 + 8740)23.67 \div 8 = 50\,300$ pound-feet. The moment at 1.25 feet from the middle of the span due to these loads is 49 750 pound-feet, whence the total maximum bending moment is $171\,750 + 49\,750 = 221\,500$ pound-feet.

Approximate theoretic determinations of the economic depth, such as that of A. JAY DUBOIS in Transactions American Society of Civil Engineers, Vol. XVI, page 191, show that the least weight depth is a little over one seventh of the span, and as the effective depth of a stringer is about one inch less than the depth of web, the depth of web on the basis of the least cost is about one ninth of the span. The depth shall preferably not be less than one tenth of the span (¶ 36) or 28.4 inches, and one ninth of the span is 31.6 inches. Further, for this panel length the economic depth of the floor beam is about the same as that of the stringer; it is also desirable that the depth of the stringer should not exceed the distance between the flange angles of the floor beam; and deepening the floor beam increases the lengths of the posts of the truss for a given clear head room. Taking all these facts into account, the web will be made 30 inches deep, giving an effective depth of about 29 inches.

The flange stress is $221\,500 \times 12 \div 29 = 91\,660$ pounds, and the net flange area required (¶ 42) is $91\,660 \div 8000$ (¶ 30) $= 11.44$ square inches. The composition of both flanges (¶ 35) is as follows (¶¶ 32, 58, 67 and 71, and Pocket Companion, pages 106 and 195 *):

| | | |
|---|---|---|
| 2 angles, $5'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$, | $2(4.00 - 0.50)$ | $= 7.00$ sq. in. |
| 1 cover plate, $12'' \times \frac{1}{2}''$, | $(12 - 2)\frac{1}{2}$ | $= 5.00$ |
| | | 12.00 |

The center of gravity of the solid upper flange is 0.41 inch below the backs of the angles; that of the net lower flange is 0.48 inch above the backs of the angles provided the section is taken through the rivet holes in the longer horizontal legs of the angles, and 0.29 inch when the section is taken through the rivet holes in the shorter vertical legs of the angles. If the angles are flush with the edges of the web, the effective depths (¶ 42) are 29.1 and 29.3 inches respectively. The net section required is reduced to 11.42 square inches for a depth of 29.1 inches. By taking into account the allowable relation of angles to web (¶ 65) the effective depth might be made such that the cover plate could be reduced to a thickness of $\frac{7}{16}$ inch. The above section will, however, be retained in this example. In the case of a single cover plate it is preferable that the cover plate should not exceed the angles in thickness even though ¶ 67 be satisfied.

The maximum shear at the end is $33\,940 + \frac{1}{2}(8260 + 8740) = 42\,440$ pounds. It is considered that a unit shearing stress equal to two thirds or three fourths of the tensile stress is allowable in the web, hence the minimum thickness of $\frac{3}{8}$ inch

* The reason why the references in this chapter are made to the Pocket Companion, which now gives tables for steel only, is that this design was almost completed before the issue of the edition of 1893, in which the tables relating to wrought iron were omitted.

allowed in the best practice will give sufficient strength. The specifications do not give the unit shear to determine web section. Applying the formula in ¶ 43, the shear allowed before stiffeners are required is 3830 pounds per square inch. The web section after deducting one rivet for each flange is 10.5 square inches and the shear at the end is 42 440 ÷ 10.5 =

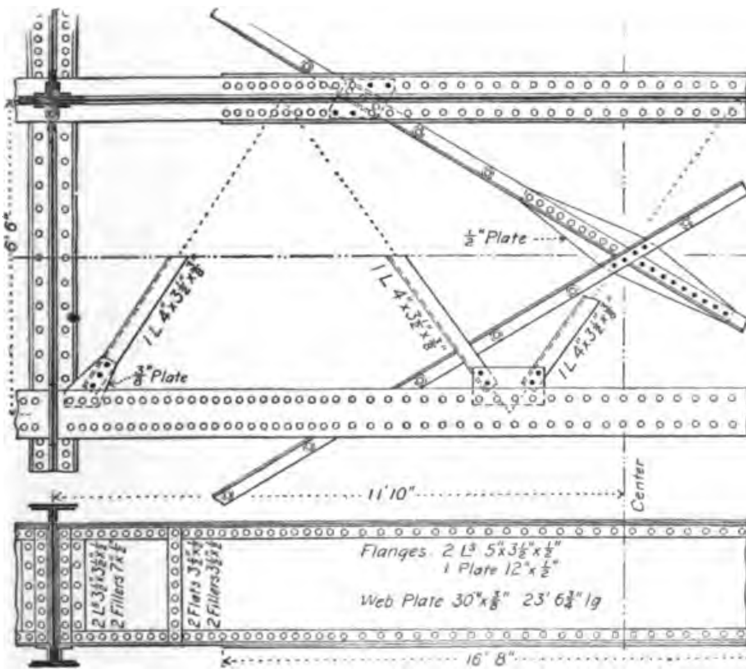



FIG. 35.

4040 pounds per square inch. Stiffeners will therefore be required at the end. The angles connecting them to the web of the floor beam will answer this purpose also. These two angles will be taken $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{2}$ inch. The duty which they have to perform is such that no definite computation can be made to determine their section. As the joints are a controlling factor in the durability of the bridge, it is essential that their parts have abundant strength. The unit shear at a section 1.5 feet from the end is within the above limit of stiffeners, nevertheless one stiffener will be placed at about 30 inches from the end to consist of one flat $3\frac{1}{2} \times \frac{1}{2}$ inch and one filler of the same section on each side of the web. The plan and elevation of the stringers and their connection with the floor beam is shown in Fig. 35.

The cover on the top flange will be continued to the end, but the lower one will be cut off where the net section will allow it. The distance of the center of gravity of the net section of the angles alone is 0.75 inch above their backs; hence the effective depth for an upper solid flange with cover and a net lower flange with angles only is $30.0 - (0.41 + 0.75) = 28.8$ inches, and the corresponding bending moment is $7.00 \times 8000 \times 28.8 \div 12 = 134\,400$ pound-feet. This ordinate is found by means of a diagram similar to Fig. 29 (Art. 36) to be at 4.1 feet from the end. After the rivet spacing is determined in the flanges the exact length of the cover plate will be found. This will now be done.

The following table contains the simultaneous bending moments expressed in pound-feet in each pair of adjacent sections three feet apart when the live load is so placed as to produce the maximum vertical shear in the section nearer the middle.

| Distance of Section from Support. | Load in Position for Maximum Shear at Section. | | | |
|---|---|---|---|---|
| | 3′ | 6′ | 9′ | 11.83′ |
| Feet. | | | | |
| 0 | 0 | | | |
| 3 | 84 000 | 65 000 | | |
| 6 | | 130 500 | 104 000 | |
| 9 | | | 148 750 | 123 750 |
| 11.83 | | | | 151 500 |
| Difference | 84 000 | 65 500 | 44 750 | 27 750 |

After adding the corresponding differences due to the dead load and wind effect the maximum differences are respectively 106 000, 81 600, 54 050 and 30 650 pound-feet. For an effective depth of 28.8 inches in the first two divisions and of 29.1 inches in the next two, the differences of flange stresses are 44 170, 34 000, 22 290 and 12 640 pounds. As these divisions are short let the weight of one driver as well as the track be considered as distributed over it. This is on the safe side as the flange stresses were obtained for the driver at the end of each division. This vertical load is 12 600 pounds. The resultants of the horizontal stresses and vertical loads are 46 400, 36 600, 25 600 and 17 900. The bearing value of a $\frac{7}{8}$-inch rivet in a $\frac{3}{8}$-inch web is 3940 pounds (¶ 39 and Pocket Companion, page 176). The average pitch of the rivets for each division (length = 36 inches) is found by dividing the product of 3940 and 36 by the resultant, the results being 3.06, 3.88, 5.54 and 7.92 inches. If these values be laid off as ordinates at the middle of each division, and a curve be drawn through their extremities, the ordinates to this curve will give the theoretic pitch at every point. Since the greater part of the weight of one driver may actually be concentrated on one cross-tie the pitch will not be allowed to exceed 4.5 inches. This pitch is found at 5.75 feet from the end, and to avoid changing the pitch more than once for the half girder, the pitch of 3 inches at the end will be continued to this section, then changed to 4.5 inches, and so continued to the middle.

The location of the rivets on the drawing makes the half length of the lower cover plate 8 feet 5 inches, if 3 rivets in each row are added beyond the theoretic limit as given above.

The number of rivets required to connect the end angles to the web of the stringer is 42 440 ÷ 3940 = 11. In order to avoid bending the end angles, fillers of the same thickness as the flange angles must be placed under them. If these fillers are not riveted to the web independently of the rivets through the angles the tendency is to bend those rivets and to overstrain the rivets at the end of the upper flange on account of the pressure of the fillers and end angles. This is avoided by widening the fillers and putting a row of rivets through them and the web outside of the angles. In this case the fillers will be $7 \times \frac{1}{2}$ inch in section. The number of rivets required in the angles alone must be at least $42\,440 \div (2 \times 4510) = 5$. The spacing of these rivets cannot be determined until the floor beam is designed, as the rivets in the legs of each connecting angle must be staggered.

Only one lateral system is required for stringers and is attached to the upper flanges. The Warren type of truss will be used (¶ 29), and if the number of panels is made $1\frac{1}{2}$, $2\frac{1}{2}$, or $3\frac{1}{2}$, the connections on both stringers will have the same relative position, which is an advantage in manufacture. As one and one half panels would in this case make the angle between the braces and stringers less than 40 degrees, two and one half panels will be taken. Taking the average depth of stringer and track as 39 inches and the wind pressure on the stringer at 30 pounds per square foot (¶ 24), the maximum stresses in the end brace are $+4470$ and $-4470$ pounds. Increasing one by eight tenths of the other (¶ 37), the sum is 8050 pounds. The length of the brace is about 90 inches, hence the unit stress in compression is $10\,500 - 60\dfrac{90}{0.73} = 3100$ pounds for $4 \times 3\frac{1}{2}$ inch angles (¶ 33 and Pocket Companion, page 104). The area is $8050 \div 3100 = 2.60$ square inches, which is furnished by one angle $4 \times 3\frac{1}{2} \times \frac{3}{8}$ inch. The same size will be used throughout. The angles will be riveted to the connecting plates by the 4-inch leg, which alone has a net section of 1.13 square inches. As the net section required by ¶ 31 is $8050 \div 15\,000$ (¶ 30) $= 0.54$ square inch, the angles satisfy this requirement. The rivets and connecting angles are shown in Fig. 35, $\frac{7}{8}$-inch field rivets being used throughout (¶¶ 39, 52, 57).

The final estimate of the weight of the stringer may now be computed with the aid of the tables in the Pocket Companion, pages 38, 40, 41, 198–200, 261–264, remembering that steel is two per cent heavier than wrought iron.

| | Pounds. |
|---|---|
| 4 angles, 5″ × 3½″ × ½″, 23′ 6¾″ long, @ 13.4 lbs. | 1262 |
| 1 cover plate, 12″ × ½″, 23′ 6¾″ long, } <br> 1 cover plate, 12″ × ½″, 16′ 8″ long, } @ 20.0 lbs. | 805 |
| 1 web plate, 30″ × ⅜″, 23′ 6⅜″ long, @ 37.5 lbs. | 883 |
| 4 flats (stiffeners), 3½″ × ½″, 29″ long, } <br> 4 fillers, 3½″ × ½″, 23″ long, } @ 5.8 lbs. | 101 |
| 4 connecting angles, 3½″ × 3½″ × ½″, 29″ long, @ 10.9 lbs. | 105 |
| 4 fillers, 7″ × ½″, 23″ long, @ 11.7 lb. | 90 |
| One half lateral system: | |
| 2½ angles, 4″ × 3½″ × ⅜″, 6′ 10″ long, @ 8.9 lbs. | 152 |
| 3 connecting plates, ⅜″ thick, 3 sq. ft., @ 15.0 lbs. | 45 |
| 462 pairs of rivet heads, @ 0.444 lb. | 205 |
| Total | 3648 |

As the connecting angles, their fillers and rivets are supported directly by the floor beam and do not affect the stresses in the stringer, the net weight of stringer forming a part of the dead load is $3648 - (105 + 90 + 10) = 3443$ pounds, which is less than the weight assumed. The gross weight per linear foot is $6.5l$ pounds, and the net weight $6.2l$. The weight of the stringer is divided as follows:

| | Weight in pounds. | Percentage of Total Weight. | |
|---|---|---|---|
| Flanges | 2067 | 56.7 | 80.9 |
| Web | 883 | 24.2 | |
| Stiffeners | 101 | 2.8 | 8.1 |
| Connection with floor beam | 195 | 5.3 | |
| Half lateral system | 197 | 5.4 | 11.0 |
| Rivets | 205 | 5.6 | |
| | 3648 | 100.0 | 100.0 |

By comparing this table with the one in Art. 46 it is seen that the lateral bracing and rivets combined have about the same relative weight. The web details, including stiffeners and connecting angles, are in this case considerably less, the sum of the flanges and web being correspondingly larger. The most noticeable difference is in the relation between the flanges and web and indicates that it is too shallow for economy when this is based only on the weight of the stringer.

In the following table are given the weights for designs made by changing the depth of web to 33 and 38 inches respectively:

| | For Web 33″ deep. | | For Web 38″ deep. | |
|---|---|---|---|---|
| | Weight. | Percentage. | Weight. | Percentage. |
| | Pounds. | | Pounds. | |
| Flanges.................... | 1818 | 51.7 | 1567 | 45.3 |
| Web........................ | 917 | 27.6 | 1118 | 32.3 |
| Stiffeners and connections (web details)............ | 324 | 9.2 | 374 | 10.8 |
| Half lateral system, and rivets.................... | 402 | 11.4 | 402 | 11.6 |
| Total............... | 3515 | 99.9 | 3461 | 100.0 |

For the web 33 inches deep the thickness of the flange angles and covers required is $\frac{7}{16}$ inch, and for the depth of 38 inches the thickness is $\frac{3}{8}$ inch. In both cases the number of stiffeners needed remains the same as before. The saving in weight in the first case is 133 pounds or 3.7 per cent, and in the second case 187 pounds or 5.1 per cent.

The weight of the flanges is approximately equal to the weight of the web and its details when the web is 38 inches deep as indicated by the table. The economic depth of the stringer, considered independently of the rest of the bridge, is therefore 38 inches.

## ART. 50. FLOOR BEAMS.

(The references to the Specifications are the same as for Art. 49.)

For the computation of stresses the span of the floor beam is taken equal to the distance between the centers of trusses, or 16 feet. As stated in the preceding Article the stringers are to be riveted to the web of the floor beam between the flange angles. For convenience in erection a bracket angle is riveted to the floor beam below the stringer. If this angle is attached by the flange rivets and it be assumed that the flange angles will be of the same size as in the stringer, the distance from the bottom of the flange angle to the top of the bracket angle (inverted) is $4\frac{1}{4}$ inches. The end depth of stringer over all is $30\frac{1}{2}$ inches. The depth of web of the floor beam must then be $30\frac{1}{2} + 4\frac{1}{4} + 3\frac{1}{2} +$ a small allowance for clearance between cover of stringer and upper flange angle of floor beam. Let the depth be taken as 39 inches. The floor beam carries, in addition to its own weight, two concentrated loads 3 feet 3 inches from its center, each load consisting of the maximum sum of the reactions of the connecting ends of the stringers on both sides, including the weight of one stringer, one half the weight of track for a panel length, the live load, and the transfer of live load due to the overturning moment of the wind, this being a release of weight on the windward and an increase on the leeward side. The weight of one stringer and the track it supports is $3650 + 4730 = 9380$ pounds. The center of pressure of the wind on the train is 7 feet above the rail and about 9 feet 3 inches above the center of the supports of the floor beams, hence the live load transferred is $300 \times 23.67 \times 9.25 \div 6.5 = 10\,100$ pounds. The maximum floor-beam reaction due to live load (see Art. 51) is 46 600 pounds. The total concentrated loads are then 45 880 and 66 080 pounds on the windward and leeward sides respectively. The bending moment will be a maximum at the leeward load, and is 285 400 pound-feet due to the concentrated loads alone. Assuming the weight of the floor beam at 2700 pounds, the moment due to this at the same point is 4510 pound-feet, making the maximum bending moment 289 910 pound-feet. If the effective depth is taken at one inch less than the depth of web the flange stress is 91 550 pounds and the net flange area required 11.44 square inches. This being exactly the same as for the stringer, the composition of the flange will be the same, the true effective depth 38.1 inches, and the revised net flange area 11.41 square inches.

The bending moment due to all external loads varies between the end and the stringer connection as the ordinates to a straight line, and the curve of moments due to the weight of the floor beam is very nearly a straight line between the same points. The rivet spacing will hence be uniform for this division. The maximum difference of flange stresses is 91 310 pounds. If the thickness of the web be taken as $\frac{3}{8}$ inch the number of rivets required will be $91\,310 \div 3940 = 23.2$, and the pitch $57 \div 23.2 = 2.46$ inches, which is below the minimum limit (¶ 50). Let the thickness of the web be increased to $\frac{7}{16}$ inch, changing the bearing value of a $\frac{7}{8}$-inch rivet to 4590 pounds (Pocket Companion, page 176) and the pitch to $2\frac{7}{8}$ inches, which is allowable. Since the entire load is transferred to the floor-beam web in a width of 11 inches, the pitch will be made 3 inches, and extended to the inner side of the stringer connection. The difference in flange stresses between the stringers is so small that the pitch of rivets derived from it exceeds the maximum limit, hence the pitch will be made 6 inches. The floor beam is shown on Plate IV and also in Fig. 35.

The cover plate of the lower flange may terminate theoretically at the point where the moment equals $7.00 \times 8000 \times 37.5 \div 12 = 175\,000$ pound-feet. This moment is at a distance from the end equal to $175\,000 \times 57 \div 289\,910 = 34.4$

inches, or $57 - 34.4 = 22.6$ inches from the stringer. The length of the lower cover plate is 11 feet 4 inches after extending it at each end to include two additional rivets in each row.

The bearing value of a $\frac{7}{8}$-inch field rivet in a $\frac{7}{16}$-inch web is $4590 \times \frac{2}{3} = 3060$ pounds (¶ 39). If field rivets alone are used to connect the end angles of the stringer to the floor-beam web, 22 rivets will be needed since $66\,080 \div 3060 = 21.6$. It is not possible to put 11 rivets in each connecting angle without using a pitch below the minimum limit. Filler plates $\frac{3}{8}$ inch thick will therefore be used on each side of the web of the floor beam. The strength of a field rivet in single shear is $4510 \times \frac{2}{3} = 3010$ pounds, hence 15 rivets must be put in the two connecting angles since $\left(33\,940 + \frac{9380 + 10\,100}{2}\right) \div 3010 = 14.5$. Strictly the resultants of these shears and the horizontal reactions of the floor beam due to the direct pressure of wind on train and stringer should be found. This would make the number of rivets 21.9 and 14.6 respectively, differing but slightly from the preceding values. As the wind pressure, however, mainly affects the upper rivets, and the deflection of the stringer has a similar effect, an excess of rivets should be provided. They will be arranged as follows: 8 rivets are to unite each angle to the floor beam, and 7 to connect the angles to the stringer web, exclusive of the two in the flanges, these two sets being staggered; 5 will be placed in the fillers on the stringers, and 7 in each row of the fillers on the floor beam. This gives 12 for the stringer web, 11 being required. Of the 30 in the floor-beam web the 14 in the fillers can be shop rivets whose combined strength is $14 \times 4590 = 64\,260$ pounds, or almost the full load. This gives the field rivets a bearing strength greater than the double shear.

Whenever the bracket angles are placed above the flange angles, the rivets in them may also be counted as part of the stringer support.

In the angles connecting the floor beam to the posts the number of rivets attaching them to each post is $(60\,080 + 1350) \div 3010 = 21$ on the basis of shear. With a pitch of 3 inches 11 can be put in each angle, and when the posts are designed it must be arranged so that the bearing strength shall be sufficient. The number of rivets connecting the angles to the web of the floor beam must be $61\,430 \div 4590 = 14$. The number of rivets in the angles alone must be at least $61\,430 \div (2 \times 4510) = 7$. As these must stagger with the preceding rows of 11 rivets, 10 will be put in the angles, excluding the two in the flanges, and 7 in the fillers. The angles are $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{2}$ inch and the fillers $7 \times \frac{1}{2}$ inch.

No stiffener will be required in the outer division of the floor beam, as the distance between the fillers will be equal to the depth of the web provided the posts are 7 inches wide, and they will probably be 9 or 10 inches wide. If a stiffener were required it would be placed diagonally across the web, from the top of the stringer to the bottom of the post connection.

The unit shear in the web may now be found (¶ 42) since the number of rivets in any section is known. The area of the section at the stringer connection is $\frac{7}{16}(39 - 9) = 13.12$ square inches, and the unit shear is $(61\,430 - 770) \div 13.12 = 4620$ pounds, the approximate weight of the segment of the floor beam outside of the stringer being 770 pounds.

Weight of one floor beam:

| | Pounds. |
|---|---|
| 4 angles, $5'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$, 15′ 2″ long, @ 13.4 lbs | 813 |
| 1 cover plate, $12'' \times \frac{1}{2}''$, 15′ 2″ long, } @ 20.0 lbs | |
| 1 cover plate, $12'' \times \frac{1}{2}''$, 11′ 4″ long, } | 530 |
| 1 web plate, $39'' \times \frac{7}{16}''$, 15′ $1\frac{7}{8}''$ long, @ 56.9 lbs | 863 |
| 4 fillers, $15'' \times \frac{5}{8}''$, 32″ long, @ 18.75 lbs | 200 |
| 4 bracket angles, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$, 14″ long, @ 10.9 lbs | 51 |
| 4 connecting angles, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$, 38″ long, @ 10.9 lbs | 138 |
| 4 fillers, $7'' \times \frac{1}{2}''$, 32″ long, @ 11.7 lbs | 125 |
| 362 pairs of rivet heads, @ 0.444 lb | 161 |
| Total | 2881 |

As the end angles and fillers are directly supported by the posts and do not affect the stresses in the floor beam the net weight of the beam is less than its assumed weight.

| | Weight in pounds. | Per cent. |
|---|---|---|
| Flanges | 1343 | 46.6 |
| Web | 863 | 29.9 |
| Web details | 514 | 17.9 |
| Rivets | 161 | 5.6 |
| Total | 2881 | 100.0 |

As the weight of the flanges is about equal to the weight of the web and its details, the total weight is about the minimum. Nearly three fourths of the rivets belong to the flanges.

## ART 51. STRESSES IN TRUSSES.

(See paragraphs 23, 24, 29, 34 and 38 of the Specifications in Art. 48.)

The following data and dimensions are tabulated below for convenient reference:

| | |
|---|---|
| Span, center to center of end pins | 142′ 0″ |
| Depth between centers of chords | 26′ 6″ |
| Width between centers of trusses | 16′ 0″ |
| Number of panels | 6 |
| Panel length | 23.667′ = 23′ 8″ |
| Length of end post, c. to c. of pins, | 35.530′ = 35′ $6\frac{11}{32}$″ |

| | |
|---|---|
| $\theta = 41° 46'$ | $\theta' = 55° 56'$ |
| tan. $\theta = 0.893$ | tan. $\theta' = 1.479$ |
| sec. $\theta = 1.341$ | sec. $\theta' = 1.785$ |

The angle which the diagonals of the truss make with the vertical is $\theta$, and that of the diagonals of the lateral bracing with the struts is $\theta'$. The opposite truss has the same letters, but primed.

The total dead load per linear foot for one truss will be taken equal to $600 + 6l$ pounds, $l$ being the span in feet. This gives a dead panel load of 17 180 pounds, of which one third, or 5730 pounds, will be applied on the upper chord, and the remainder at the panel points of the lower chord.

The specified wind load for the upper lateral bracing (¶ 24) will be considered as equally divided between the windward

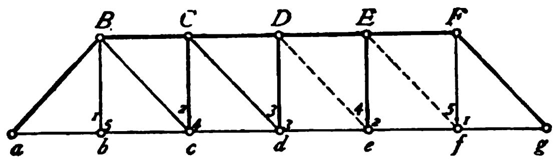



FIG. 36.

and leeward chords, giving a wind panel load of 1775 lbs. for each chord. For the lower lateral system the same panel load on each chord is treated as a fixed load (the wind pressure on the truss), and the wind pressure on the train is similarly divided and treated as a moving load (¶ 24). The latter panel loads are 3550 pounds.

The wind panel loads on the upper chord not only strain the lateral system, but tend to overturn the bridge. The lateral system transfers the intermediate panel loads to the windward ends of the portal struts. The axis of rotation passes through the supports of the leeward truss. As the center of pressure on the upper half of the bridge is really somewhat below the upper chord and the support is about a foot below the lower chord, the lever arm of the wind load will be taken equal to the depth of the truss, or 26.5 feet. The overturning moment is then $5 \times 1775 \times 26.5 = 235\,190$ pound-feet for each half of the bridge. The same stresses in the trusses will be reproduced if the wind loads are replaced by two couples each one consisting of an upward force applied at the windward end of the portal and a downward force at its leeward end, each force being equal to $235\,190 \div 16 = 14\,700$ pounds. These loads will cause stresses in the chords and end posts only.

The wind pressure on the train likewise tends to overturn the bridge about the same axis. The vertical distance from the center of pressure to the axis is estimated at 12.5 feet. The stresses in the trusses will be the same if these wind loads be replaced by a vertical load at each lower panel point equal to $2 \times 3550 \times 12.5 \div 16 = 5550$ pounds, upward on the windward, and downward on the leeward, truss. It must be remembered that these overturning moments of the wind always act in conjunction with the dead and live loads, and that they practically transfer some of these loads from the windward to the leeward trusses. The stresses in both trusses due to this cause are then equal in magnitude but opposite in character. For a freight car the center of pressure of the wind is about 7 feet above the rail, and for a wind pressure of 300 pounds per linear foot the car must weigh over 860 pounds to avoid being turned over. The 35-foot box cars of 60 000 lbs. capacity of the Michigan Central Railroad weigh 27 850 pounds when empty, and a pressure of 280 pounds per linear foot would overturn them.

The stresses in the trusses and lateral systems are given in the following tables:

| | End Post. | Upper Chord. | | Lower Chord. | | |
|---|---|---|---|---|---|---|
| | $aB$ | $BC$ | $CD$ | $ab$ | $bc$ | $cd$ |
| **Dead load** | − 57 600 | − 61 400 | − 69 000 | + 38 400 | + 38 400 | + 61 400 |
| **Live load** | −132 100 | −134 900 | −154 200 | + 88 000 | + 88 000 | +134 900 |
| **Wind overturning:** | | | | | | |
| **On truss, east** | − 19 700 | − 13 100 | − 13 100 | + 13 100 | + 13 100 | + 13 100 |
| **On truss, west** | + 19 700 | + 13 100 | + 13 100 | − 13 100 | − 13 100 | − 13 100 |
| **On train, east** | − 18 600 | − 19 800 | − 22 300 | + 12 400 | + 12 400 | + 19 800 |
| **On train, west** | + 18 600 | + 19 800 | + 22 300 | − 12 400 | − 12 400 | − 19 800 |
| **Wind on truss, east** | | + 0 | + 7 900 | + 0 | + 13 100 | + 21 000 |
| **Wind on truss, west** | | − 7 900 | − 10 500 | − 13 100 | − 21 000 | − 23 600 |
| **Wind on train, east** | | | | + 0 | + 26 300 | + 42 000 |
| **Wind on train, west** | | | | − 26 300 | − 42 000 | − 47 300 |
| **Maximum stress** | − 228 000 | −229 200 | −250 700 | +151 900 | +191 300 | +292 200 |
| **Minimum stress** | − 37 900 | − 56 200 | − 66 400 | + 12 200 | + 4 300 | + 24 700 |

| | Main Ties. | | Counters. | | Verticals. | | |
|---|---|---|---|---|---|---|---|
| | *Bc* | *Cd* | *De* | *Ef* | *Bb* | *Cc* | *Dd* |
| Dead load......... | + 34 500 | + 11 500 | − 11 500 | − 34 500 | + 11 500 | − 14 300 | [− 5 700] + 2 900 |
| Live load....... | + 86 900 | + 50 300 | + 23 700 | + 6 200 | + 46 600 | − 37 500 | − 17 700 |
| Wind overturning: | | | | | | | |
| On train, east.... | + 12 400 | + 7 400 | + 3 700 | + 1 200 | + 5 600 | − 5 600 | − 2 800 |
| On train, west... | − 12 400 | − 7 400 | − 3 700 | − 1 200 | − 5 600 | + 5 600 | + 2 800 |
| Maximum stress... | +133 800 | + 69 200 | + 25 900 | 0 | + 63 700 | − 57 400 | − 17 600 |
| Minimum stress ... | + 27 100 | 0 | 0 | 0 | + 11 500 | − 5 700 | − 5 700 |

| | Upper Lateral Bracing. | | | | |
|---|---|---|---|---|---|
| | Diagonals. | | Struts. | | |
| | *BC'* | *CD'* | *BB'* | *CC'* | *DD'* |
| Wind on truss.............. | + 9500 | + 3200 | − 2700 | − 3600 | − 1800 |
| Initial tension.............. | | | − 4200 | − 8400 | − 8400 |

| | Lower Lateral Bracing. | | | | | |
|---|---|---|---|---|---|---|
| | Diagonals. | | | Struts. | | |
| | *ab'* | *bc'* | *cd'* | *bb'* | *cc'* | *dd'* |
| Wind on truss.............. | + 15 800 | + 9 500 | + 3 200 | − 7 100 | − 3 600 | − 1800 |
| Wind on train............. | + 31 700 | + 21 100 | + 12 700 | − 15 400 | − 10 600 | − 7200 |
| Total................. | + 47 500 | + 30 600 | + 15 900 | − 22 500 | − 14 200 | − 9000 |

As the counter *Ef* is not required, the stresses in it with their signs changed will occur in *Bc* when the live load comes on from the left, making the minimum stress in *Bc* equal to $+ 34\,500 - 6200 - 1200 = + 27\,100$ pounds. COOPER'S Specifications do not require the minimum stresses in proportioning the members, but they are given here to show the ranges of stress.

The initial tension in the upper lateral struts is computed as required by ¶¶ 34 and 96. If the area of the adjacent diagonals were determined by the unit stress of 15 000 pounds (¶ 30) it would in this example give rods less than $\frac{3}{4}$ square inch in section, hence the initial tension in each of them will be 7500 pounds.

In Art. 31 was described the graphic method of finding, by means of a load line, the position of the live loads producing maximum moment. As there stated, the same method and criterion apply alike for trusses and girders. The values of the moments for trusses are usually obtained without drawing the closing lines, by reading off the ordinate at the right support, multiplying this by the ratio $l' \div l$, and subtracting the ordinate at the center of moments whose value was marked on the moment diagram when it was constructed and hence need not now be read by scale. For instance, the member $BC$ (Fig. 36) has its center of moments at $c$. When wheel 8 is at $c$ the live load will produce the maximum moment. Placing the truss diagram (similar to that in Fig. 23) over the moment diagram (similar to Fig. 22) the ordinate at the right support reads 15 550 thousand pound-feet, and the ordinate at $c$ is 1607.4 thousand pound-feet. The moment is then found to be $(\frac{1}{3} \times 15\,550) - 1607.4 = 3575.9$ thousand pound-feet, and the stress in $BC = 3575.9 \div 26.5 \times 1000 = 134\,900$ pounds.

In Part I, Art. 60, the criterion for the position of the live load producing the greatest shear in any section of the truss was found to be $P' = \frac{1}{m} W$, in which $W$ is the whole load on the truss, $P'$ the wheel loads (one or more) on the panel cut by the section, and $m$ the number of panels in the truss. Let this equation be transformed into $W = mP'$. When the truss diagram is placed on the load line the value of $W$ can at once be read off at the right support for any given position of the truss with respect to the loads. If wheel 2 be placed just on the right of a panel point $W$ must equal $mP_1$, and if just a little to the left of the same panel point then $W$ must be

$m(P_1+P_2)$. The condition will therefore be satisfied if wheel 2 is at the panel point and the value of $W$ is found to lie between the values $mP_1$ and $m(P_1+P_2)$, and similarly for any other wheel. In this example $m=6$, $P_1=7500$ pounds, and $P_2=P_3=P_4=12\,000$ pounds per truss. The following table is then arranged:

| Wheel at Panel Point. | Corresponding Values of $W$. |
|---|---|
| 1 | 0— 45 000 |
| 2 | 45 000—117 000 |
| 3 | 117 000—189 000 |

By placing a wheel at a given panel point it can be seen at a glance whether the position satisfies the condition.

If $M_1$ is the ordinate to the equilibrium polygon at the right support, $M_2$ that at the right end of the panel cut by the section, and $p$ the panel length, the vertical shear in the section is

$$V=\frac{M_1}{l}-\frac{M_2}{p}=\frac{M_1-mM_2}{l}.$$

The values of $M_1$ and $M_2$ are read off from the diagram. For the post $Cc$ the wheel 3 is at $d$, $M_1=6360$, $M_2=172.7$, both in thousand pound-feet, $m=6$ and $l=142$ feet; whence $V=(6360-6\times172.7)\div142\times1000=37\,500$ pounds, and the stress is $-37\,500$ pounds.

The maximum floor-beam reaction or stress in the suspender $Bb$ due to live load is conveniently found as follows: In Fig. 36 let $R_a$ be the stringer reaction at $a$, and $R_b$ the sum of the adjacent stringer reactions, or the floor-beam reaction, at $b$. Let $P$ be the whole load on the two equal panels $ab$ and $bc$, and $g$ the distance of its center of gravity from $c$; let $P'$ be the load on $ab$, and $g'$ the distance of its center of gravity from $b$. Since the sum of the moments of loads and reactions about $b$ is zero,

$$R_ap-P'g=0, \quad \text{or} \quad R_ap=P'g'.$$

Taking moments about $c$,

$$R_a 2p + R_b p - Pg = 0.$$

Substituting and reducing,

$$R_b = \frac{Pg - 2P'g'}{p}.$$

If the loads be moved a distance $dx$ to the left, both $g$ and $g'$ will receive an increment $dx$, and $R_b$ an increment

$$dR_b = \frac{Pdx - 2P'dx}{p}.$$

Placing the derivative equal to zero gives

$$P = 2P'.$$

That is, when the live load in both panels is double that in the panel $ab$ the resulting value of $R_b$ is a maximum. This is the same condition as for finding the maximum bending moment at the middle of the girder whose span is $ac$.

The use of the load line gives the position very quickly, and the moments $Pg$ and $P'g'$ can be read off on the moment diagram. If $M_c$ be the moment ordinate at $c$, and $M_b$ that at $b$, the value of $R$ may be more conveniently expressed and remembered as

$$R_b = \frac{M_c - 2M_b}{p}.$$

In the case of bridges where the two panels at the end, $p_1$ and $p_2$, are not equal the value of $R_b$ deduced in a similar manner is

$$R_b = \frac{p_1 M_c - (p_1 + p_2)M_b}{p_1 p_2},$$

the criterion for loading being that which produces a maximum moment at $b$ in a girder whose span is $ac$, $p_1$ being the length $ab$, and $p_2$ the length $bc$.

The transverse or sway bracing of the trusses will consist in uniting the intermediate posts to the lateral struts by a bracket having a solid web (see Plate IV). An outline diagram is given in Fig. 37. As the posts are fixed at both ends, so far as transverse flexure is concerned, their points of inflection are midway between the floor beam and the bracket, or about 13.5 feet from $D$ and $D'$. The horizontal reactions may therefore be considered as applied at those points. The vertical reactions equal $2 \times 1775 \times 26.5 \div 16 =$ 5880 pounds. The bending moment to be resisted by the bracket is $M = 1775 \times 13.5 = 23960$ pound-feet. The maximum bending moment in the post is at $L$ and equals $1775 \times 9 = 15975$ pound-feet. That in the strut $DD'$ is at $K$ when the forces on the left are considered and equals $1775 \times 13.5 - 5880 \times 11.5 =$ 43 660 pound-feet. The direct stresses in the members are given in the preceding table. The computations just made assume that the upper lateral system is not acting. This assumption is usually made to furnish some convenient basis for ascertaining the additional stresses to be used in proportioning the members which are employed to secure the transverse stiffness of the bridge.

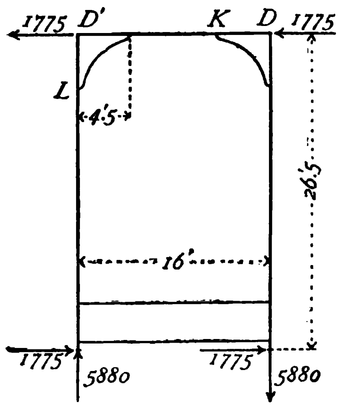



FIG. 37.

The clearance at 20 feet above the base of the rail is generally specified as 6 feet wide, and the portal bracket may be taken so that the vertical projection of a tangent to its inner edge has a slope of about 45 degrees.

It is estimated that the lower end of the portal strut is about 8.0 feet from the upper pin of the inclined end post. The post is 35.53 feet long. It is proposed to use an end floor beam and to attach it rigidly to the end post. This will extend to a point about 4.5 feet above the lower end of the post. If the point of inflection be taken half way between these points it will be about 19.5 feet from the upper end of the post. The wind load is 7100 pounds at the windward end of the portal strut and 1775 at the leeward end. Assuming both supports to react equally, each horizontal reaction is 4437.5 pounds, and the reactions coinciding with the axes of the end posts are equal to $8875 \times 35.53 \div 16 = 19\,708$ pounds. The maximum bending moment in the end post is then $4437.5 \times (19.5 - 8.0) = 51\,030$ pound-feet. The bending moment in the portal strut is a maximum at the end of the portal bracket, which may be taken at 3.5 feet from the middle of the strut. At this point its value is $4437.5 \times 19.5 - 19\,708 \times 11.5 = 140\,110$ pound-feet.

The stress in the end lower lateral strut at the free end of the bridge equals the horizontal reaction of the windward shoe due to the wind loads, minus the friction of this shoe, the coefficient of friction being taken at one fourth. If the adjacent lower lateral were adjustable the component of its initial tension would also be added. The vertical load on the shoe is the weight as reduced by the overturning moment of the wind.

When the bridge is empty the stress in this strut *aa'* (Fig. 36) is $(2\frac{1}{2} \times 1775) + (5\frac{1}{2} \times 1775) - \frac{1}{4}(2\frac{1}{2} \times 5730 + 3 \times 11\,450 - 14\,700) = 14\,200 - 8490 = 5710$ pounds, and when a train weighing 860 pounds per foot covers the bridge the stress is $5710 + 5\frac{1}{2} \times 3550 - \frac{1}{4}(30\,530 - 3 \times 5550) = 21\,770$ pounds. At the fixed end the stress may be taken at one half the shear in the end diagonal of the lower lateral system or $2\frac{1}{2}(1775 + 3550) = 13\,310$ pounds.

## ART. 52. SECTIONS OF INTERMEDIATE POSTS.

(See paragraphs 33, 41, 47 and 85 of the Specifications in Art. 48.)

The column formulas given in ¶ 33 are to each other as 1, 2, and 1.5 for live, dead and wind stresses respectively.

It will therefore be most convenient to reduce the stresses to equivalent live-load stresses, and use the corresponding formula. The total equivalent live-load compression in the post $Cc$ is

$$\tfrac{1}{2}(14\,300) + 37\,500 + \tfrac{2}{3}(5600) = 48\,400 \text{ pounds.}$$

The length of the post is 26.5 feet, or 318 inches. Let a section be tried consisting of two 10-inch channels with their webs parallel to the plane of the truss. The pocket book gives the following data concerning a form which is rolled to any weight between the limits given:

| | Weight. | Area. | Moment of Inertia. $I$. | Radius of Gyration. $r$. |
|---|---|---|---|---|
| | 35 | 10.5 | 121.2 | 3.40 |
| | 17.5 | 5.2 | 77.0 | 3.83 |
| Difference, | 17.5 | 5.3 | 44.2 | 0.43 |

Let the value of $r = 3.7$ be tried. This gives an average equivalent live-load unit stress for the post of

$$7000 - 40\,\frac{318}{3.7} = 3560 \text{ pounds}$$

and an area for one channel of $\tfrac{1}{2}(48\,400 \div 3560) = 6.8$ square inches.

By interpolation the elements corresponding to an area of 6.8 square inches are: weight, 22.8; $I$, 90.3; and $r$, 3.70. As this value of $r$ agrees with that assumed no modification is necessary. The flanges of the channels will be turned inward as this gives the most economical section. The moment of inertia $I'$ of one channel with respect to its neutral axis parallel to the web is 3.2, the center of gravity being 0.67 inch from the outside of its web. Let $h$ be the distance from this neutral axis to a parallel axis through the center of the posts, with respect to which the channel is to have a moment of inertia equal to $I$, and since $I = I' + Ah^2$, in which $A$ is the area of the channel, $90.3 = 3.2 + 6.8h^2$, whence $h =$ 3.58 inches. The post would be equally strong in both directions under the given direct stresses when the distance back to back of channels is $2(3.58 + 0.67) = 8.5$ inches, provided the length of the column were the same for flexure in both directions. For bending transversely with respect to the truss, the effective length is the clear distance between the floor beam and the bracket, since the latter has a solid web. But the post is to be designed to take a bending moment of 15 975 pound-feet at a point 4.5 feet below its upper extremity (Art. 51). As the column formulas in ¶ 33 are not adapted for use in combining in a rational manner the unit stress in the outer fiber of a column due to the longitudinal pressure and the flexural unit stress produced by a lateral force, some other method must be employed. It will be on the side of safety to disregard the difference in lengths just referred to and to make the moment of inertia $I_2$ of the post equal to the sum of its moment of inertia $I$ considered as a column, and its moment of inertia $I_1$ when treated as a beam subject to the above bending moment (¶ 41).

$$I_1 = \frac{Mc}{S} = \frac{15\,975 \times 12 \times 5.1}{10\,500} = 91.3,$$

$c$ being one half the assumed distance back to back of the post channels (and which must be somewhat larger than one half of 8.5 inches), and $S$ the allowed unit stress in the outer fiber due to wind (¶ 33). $I_1$ for one channel is $93.1 \div 2 = 45.6$, and

$$I_2 = I + I_1 = 90.3 + 45.6 = 3.2 + 6.8h^2,$$

whence $h = 4.44$ inches. The value of $c$ is then $4.44 + 0.67 = 5.11$ inches, which agrees with the value assumed above. The distance back to back of the post channels is hence $10\frac{1}{4}$ inches, but considering the difference of length mentioned above and the fact that the bending moment occurs at a distance of nearly 9 feet from the middle of the post, the distance may safely be reduced to 10 inches.

If the webs of the channels were placed perpendicular to the plane of the truss the required moment of inertia for one channel would be $90.3 + 45.6 = 135.9$, which exceeds that of the largest 10-inch channel.

For the post $Dd$ the required area of one channel is 2.6 square inches for $r = 3.62$. This is less than the smallest 10-inch channel listed in the handbook, but this shape will be taken. The corresponding value of $I$ is $2.6(3.62)^2 = 34.1$. Assuming $c = 4.5$ inches, $I_1 = 42.0$ for one channel, $I_2 = 75.2$ and $h = 3.90$, whence $c = 3.90 + 0.55 = 4.45$ inches, which agrees closely enough with its assumed value. The backs of channels in this post must then be placed 9 inches apart, or farther if other considerations require it. The weight of these channels is 16 pounds per foot. If 9-inch channels are tried, the following results are obtained for one channel of each post. For $Cc$: weight $= 26.7$, area $= 8.0$, $I = 78.0$, $r = 3.19$, $I' = 3.7$, $I_2 = 119.1$, $c = 4.50$; and for $Dd$: area required $= 2.86$, $I = 31.5$, actual area of least section $= 4.3$, weight $= 14.5$, $r = 3.32$, $I' = 2.0$, $I_2 = 74.4$, $c = 4.68$.

The comparative weights for one truss are:

For 10-inch channels: $2 \times 22.8 + 16.0 = 61.6$ pounds.

For 9-inch channels: $2 \times 26.7 + 14.5 = 67.9$ pounds.

As the latter size would require about 10 per cent more iron than the former, the 10-inch channels will be used. Their webs have a thickness of 0.45 and 0.32 inch, and their flanges are 2.64 and 2.51 inches wide respectively. The arrangement of the lattice bars, batten, or stay plates, pin plates and other details of the posts will be treated under separate topics later. It may be stated here, however, that the inside width of channels required by the angles and diaphragm or web uniting them at the connection with the floor beam must also be considered in some cases in selecting their size.

## ART. 53. SECTIONS OF TIES AND OF LOWER CHORD.

**(See paragraphs 30, 32, 38, 41, 45 and 79 of the Specifications in Art. 48.)**

Most of the tension members of a pin-connected truss are made in the form of eyebars. The ratio of the thickness to the width of the body of the bar varies from about $\frac{1}{8}$ to $\frac{1}{5}$, the latter ratio being approached as the number of bars in the same panel increases.

The equivalent live-load stress for the main tie *Bc* is $\frac{1}{2}(34\,500) + 86\,900 + \frac{2}{3}(12\,400) = 112\,400$ pounds, and the area required $112\,400 \div 8000$ (¶ 30) $= 14.05$ square inches. Two eyebars $5 \times 1\frac{7}{16}$ inches give 14.38 square inches (Pocket Companion, page 193). Similarly the area required for *Cd* is 7.63 square inches and the section may consist of two bars $5 \times \frac{13}{16}$ inch $= 8.12$ square inches, or two bars $4 \times 1$ inch $= 8$ square inches. The latter section has the better proportion and will be used. Decreasing the width of diagonals toward the middle of the truss also gives a better appearance than if all had the same width. The stresses in the counter *De* ($=$ *Dc*) require an area of $(-\frac{11\,500}{2} + 23\,700 + \frac{2}{3} \times 3700) \div 8000 = 2.55$ sq. in. (¶ 45). The "counter live load" referred to in the specifications would require 2.96 square inches, but the minimum area is to be 3 square inches. The section of counters is preferably square and they have to be upset for the sleeve nuts used to adjust them. One bar $1\frac{3}{4}''$ square gives an area of 3.06 square inches, or two bars $1\frac{1}{4}''$ square give 3.12 square inches (Pocket Companion, page 203). The single bar will be used.

In order to reduce the effect of impact it is desirable to make the suspender *Bb* a member capable of resisting compression as well as tension, thus giving it the composition of a post. Its net section is 7.48 square inches, the unit stress being less for shapes than for eyebars (¶ 30). Its connection with the floor beam requires two rivet holes to be deducted from the section of each channel. The lightest 10-inch channel has a web 0.32 inch thick, giving a net section for each channel of $4.8 - 0.64 = 4.16$ square inches. Above the floor beam connection the rivets are in the channel flanges, giving a net section of $4.8 - 0.70 = 4.1$ square inches. These channels weigh 16 pounds each per foot, and will be used. In a short span like this it is desirable to have all the posts composed of the same depth of channel to secure uniformity in the details.

The minimum stress in the lower chord *bc* is only 4300 pounds. If adjustable rods were used for lower laterals, the component of their initial tension would very probably exceed this amount and so cause compression. As it is proposed to use angle irons for laterals that reason will not apply in this case, but in order to increase the rigidity of this part of the bridge the lower chord in the first and second panels will be made a continuous member capable of taking compression. A convenient section consists of two flats and four angles united by lattice bars along the neutral surface. This arrangement has an advantage in a tension member in having the area of the angles near the neutral surface, while if channels were used a considerable part of this area is farther away. The combined wind stresses in *bc* amount to 64 900 pounds, or more than half of the sum of the dead- and live-load stresses (¶ 38). The area for dead and live load only is

$$\frac{38\,400}{15\,000} + \frac{88\,000}{7500} = 14.30 \text{ square inches,}$$

making the average unit stress $(38\,400 + 88\,000) \div 14.3 = 8840$ pounds per square inch. Increasing this by 25 per cent, the net area required is $191\,300 \div 11\,050 = 17.32$ square inches. The weakest section has two rivet holes in each flat and one in each angle. The composition of the section is then as follows:

| | | |
|---|---|---|
| 2 flats, $12'' \times \frac{1}{2}''$, $2(6.0 - 1.0)$ | = | 10.00 sq. in. |
| 4 angles, $3\frac{1}{2}'' \times 3'' \times \frac{3}{8}''$, $4(2.30 - 0.375)$ | = | 7.70 |
| Total net area.................... | | 17.70 |

For the chord *cd* an area of 24.65 square inches is required. Two eyebars $6 \times 1\frac{1}{16}$ inches, and two bars $6 \times 1$ inch will give 24.76 square inches, or 4 bars $5 \times 1\frac{1}{4}$ inches will give 25.00 square inches (Pocket Companion, page 193). The bars 6 inches wide will pack into a chord $1\frac{1}{8}$ inches narrower than the 5-inch bars (¶ 79), but when the stress due to their weight is taken into account (¶ 41) it will require an increase of section for the 6-inch bars, but not for the 5-inch bars. The unit stress used was 11 850 pounds per square inch, which when increased 10 per cent amounts to 13 035 pounds. The unit stress due to the weight of the bar for a depth of 5 inches is 1250 pounds (Art. 18). Hence the actual unit stress is $(292\,200 \div 25.00) + 1250 = 12\,940$ pounds per square inch, which is within the allowable limit. For the 6-inch bars the unit stress is $(292\,000 \div 24.76) + 1280 = 13\,080$ pounds, which exceeds the limit. Four bars $6 \times 1\frac{1}{16}$ inches with a sectional area of 25.52 square inches would therefore be required to satisfy the condition of ¶ 41. The 5-inch bars will be selected since they are lighter and also since it will reduce the number of widths of eyebars to be manufactured.

## ART. 54. PINS AND EYEBAR HEADS.

(See paragraphs 40, 73, 77, 78 and 79 of the Specifications in Art. 48.)

In trusses of short spans the pins are made of the same diameter throughout, the advantage of uniformity in details, depending upon them, more than counterbalancing the additional material required in the pins themselves. The allowed unit stresses for shear and bearing in ¶ 40 refer to all members excepting eyebars, while the first part of ¶ 79 refers to eyebars. The size of the pins will be determined by that required at the panel point *d*.

The stresses in all the members meeting at *d* to be used in finding the bending moment in the pin must be simultaneous stresses, the position of the load being that which produces the maximum moment in the truss at the panel point *d*. The required position of the live load places wheel 12 (second driver of second engine) at *d*, the live-load stresses being as follows: In *cd*, 134 400; in *de*, 133 700, in *Cd*, 29 700; and in *Ed*, 30 800 pounds. The stress in *Cd* due to the overturning moment of the wind on the train for a load covering the whole bridge is 3700 pounds, and that in *Ed* is the same. The stresses due to other loads remain the same as given in the table in Art. 51. The horizontal components of the total stresses in the diagonals, for this position of the live load, are: *Cd*, 29 900, and *Ed*, 30600 pounds; while their vertical components are 33 500 and 34 300 pounds respectively. The corresponding pressure of the post on the pin is 67 800 pounds, and the stresses in the chords *cd* and *de* are 291 700 and 291 000 pounds respectively.

The horizontal forces in the chords and diagonals produce flexure in the pin in a horizontal plane, the bending moment being designated by $M_h$, while the vertical forces in the diagonals and post produce a moment $M_v$, their resultant being $M = \sqrt{M_h^2 + M_v^2}$. In order to reduce $M_v$ the eyebars of the diagonals are placed next to the post. The chord bars alternate in direction, the one next to a diagonal having the same direction as the latter. The arrangement, or packing, of one half of the eyebars and shapes meeting at *d* is indicated in Fig. 38. A clearance of $\frac{1}{16}$ inch is allowed between eyebars, and $\frac{1}{8}$ inch between an eyebar and the post. The straight sides of the equilibrium polygon are drawn by considering the stresses applied at the centers of the bearings against the pin (¶ 40), while the curved lines show the form of the moment diagram when the stresses are regarded as uniformly distributed on the bearings. The curves are parabolas, the points of tangency lying in the verticals through the sides of the eyebars (Part II, Art. 10). The diagram was drawn full size for the linear measurements, and the pole distance made equal to 50000 pounds. The ordinate inside of the post bearings (where $M_v$ is a maximum) measured 3.51 inches, making $M_h = 3.51 \times 50000 = 175\,500$ pound-inches. $M_v = 17\,150 \times 2.22 + 16\,750 \times 1.16 = 57\,600$ pound-inches, whence $M = 184\,700$ pound-inches. Referring to the Pocket Companion, page 173, it is found that an iron pin $5\frac{1}{16}$ inches in diameter is needed, or a steel pin $4\frac{3}{4}$ inches in diameter (¶ 118), the unit stress for the latter being 18000 pounds per square inch. As pins are manufactured in diameters varying by $\frac{1}{8}$ inch and must

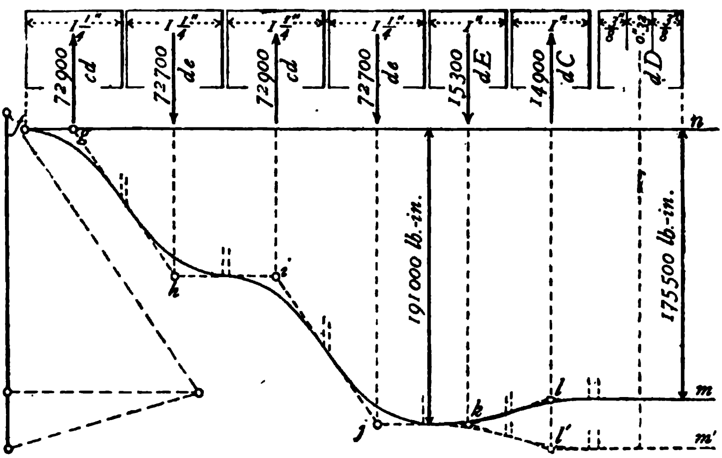


FIG. 38.

be finished by turning them down $\frac{1}{16}$ inch, the iron pin would be finished from a diameter in the rough of $5\frac{1}{8}$ inches and the steel pin from $4\frac{7}{8}$ inches. The maximum value of $M_h$, however, is given by the ordinate at the outer edge of the diagonal $dE$, which measured 3.82 inches. This makes $M_h = 3.82 \times 50000 = 191\,000$ pound-inches, which also equals $M$, since at this point $M_v = 0$. This then is the maximum value of $M$, which requires a $4\frac{13}{16}$-inch steel pin, and this finished size will be selected. It will be observed that this diameter exceeds two thirds of the depth of the largest eyebar (¶ 79).

If the position of the diagonals had been reversed, placing one adjacent to a chord bar extending in the opposite direction, the value of $M_k$ would have been increased to 208 500 and $M$ to 216 300 pound-inches, thus requiring pins of $\frac{1}{4}$ inch larger diameter. The corresponding change in the equilibrium polygon is shown by the broken line $k'l'm$ in Fig. 38.

Were the stresses in the diagonals disregarded and the maximum stresses in $cd$ and $de$ (= 292 200 pounds) considered simultaneous, the resulting bending moment in the pin would be 191 800 pound-inches, which requires a steel pin $4\frac{13}{16}$ inches in diameter.

If the allowed bearing stress of 12 000 pounds were applied to eyebars the pressure per linear inch in one of the four eyebars in $cd$ would be $(292\,200 \div 4) \div 1.25 = 58\,440$ pounds, which would require a $4\frac{7}{8}$-inch pin (Pocket Companion, page 174).

If the eyebars were 6 inches deep instead of 5 inches, the maximum bending moment on the pin would be 165 500 pound-inches, the corresponding diameters of pins being $4\frac{7}{8}$ inches for iron and $4\frac{9}{16}$ inches for steel. The entire joint would be $1\frac{1}{2}$ inches narrower than for 5-inch bars. These results are given simply to show the comparative effect of changes in the dimensions of the parts connected by the pin.

The section of the head of the eyebar depends upon the ratio of the diameter of the pin to the width of the bar and upon the radius of the curve of its neck, as well as upon the relative tensile and compressive strengths of the material of which it is composed. The radius of the neck should be large enough to transmit the stress by an easy curve, and should not be less than the diameter of the head. Specifications usually require (¶ 73) that the bar, if tested to destruction, shall break in the body of the bar and not in the head. The excess of section in the head over that in the body varies from 30 to 50 per cent, though generally the higher limit is about 40 per cent. These values are the result of numerous experiments. In the present example the excess will be taken at 37 per cent, the crown being concentric with the eye or pin hole. The section of the head of a 5-inch bar is then $5 \times 1.37 = 6.85$ inches wide and of the same thickness as the bar. With a clearance of $\frac{1}{32}$ inch (¶ 78) the diameter of the head will be $6.85 + 0.031 + 4.813 = 11.694$ or $11\frac{3}{4}$ inches. For the bars 4 inches wide the diameter of the head must be $10\frac{3}{8}$ inches. The counter ties have loop eyes at the ends, the section of the loop being uniform throughout and the same as that of the body of the bar. The distance from the center of the pin to the inner end of the loop should be twice the diameter of the pin.

## ART. 55. UPPER-CHORD SECTIONS.

(See paragraphs 33, 38, 41, 70 and 90 of the Specifications in Art. 48.)

The upper chord usually consists of two vertical web plates with an angle iron at the outer top and bottom of each web, the whole united by a cover plate above and by latticing below. The web must be deep enough so that the head of the eyebar *Bc* which is placed on the outside of the chord will clear the under side of the angle and also its rivets. Adding the radius of the pin, the half width of section of eyebar head, height of rivet head and thickness of angle, the result is $2.41 + 3.43 + 0.64 + 0.38 = 6.86$ inches. (See Fig. 9.) As the span is short it is assumed that no thicker angle will be needed than $\frac{3}{8}$ inch. The web ought then to be 14 inches deep. The clearance between the webs will depend upon the members at the panel point *C*. The post *Cc* measures 10 inches back to back of channels, and its webs need pin plates whose thickness for moderate spans should rarely vary much from $\frac{5}{8}$ inch. An allowance for the space between shapes with countersunk rivets may be taken at $\frac{1}{8}$ inch. The tie *Cd* is 1 inch thick.

The half clearance needed is then $5.0 + 0.375 + 0.125 + 1.0 + 0.125 = 6\frac{5}{8}$ inches. If the distance between webs be made 14 inches the rivet heads may be flattened instead of countersunk.

As pin plates should be placed outside of the vertical legs of the angles no smaller size can be used than $3\frac{1}{2} \times 3$ inches, the longer side being placed horizontal. As some allowance must be made for the thickness of webs the cover plate will be 22 inches wide.

The pins are placed in the neutral axis of the chord section or else secondary stresses will be caused. Convenience in manufacture also makes it desirable that the pins shall be in the center line of the pin plates. If the upper and lower angles have the same size the pin plates and their fillers will fit well and the pins will be brought to the mid depth of the webs. To bring the center of gravity to this position flats are riveted to the horizontal leg of the lower angles of about double the thickness of the cover. Their required width is readily computed. The form of chord section is shown in Fig. 39.

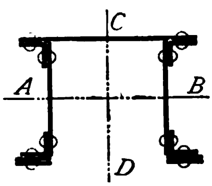


FIG. 39.

The clear width between the rows of rivets in a web 14 inches deep with 3-inch vertical legs of angles is $14 - (2 \times 1.75) - 1\frac{7}{16} = 9\frac{1}{16}$ inches (Art. 15), and therefore the thickness must be at least $\frac{5}{16}$ inch (¶ 70). The allowable clear distance between the rivet lines of a $\frac{7}{16}$-inch cover is $40 \times \frac{7}{16} = 17.5$ inches, and the actual distance will be within this limit provided the web does not exceed $\frac{7}{16}$ inch in thickness.

The allowable unit stress (¶ 33) involves the radius of gyration of the section to be designed. An approximate value may be found by taking four tenths of the depth over all. In this case $r = 0.4\ (0.44 + 14.0 + 0.88) = 6.13$. The length of the chord is 23 feet 8 inches = 284 inches, and the live-load unit stress is

$$8000 - 30\frac{284}{6.13} = 6610 \text{ pounds per square inch.}$$

As the stresses in the upper chord due to wind are less than 25 per cent of the sum of the dead- and live-load stresses they need not to be taken into account (¶ 38). The equivalent live-load stress in $CD$ is $\frac{1}{2}(69\,000) + 154\,200 = 188\,700$ and the area required, $188\,700 \div 6610 = 28.55$ square inches. It is specified in ¶ 90 that the material must mostly be concentrated at the sides, in the angles and webs. If the angles and webs are made as heavy as the cover the area will be very much greater than required. If no thinner iron be employed than $\frac{3}{8}$ inch the composition of the section and the resulting area are as follows

| | | |
|---|---|---|
| 1 cover plate, | $22'' \times \frac{7}{16}''$, | 9.62 sq. in. |
| 4 angles, | $3\frac{1}{2}'' \times 3'' \times \frac{3}{8}''$, | 9.20 |
| 2 webs, | $14'' \times \frac{3}{8}''$, | 10.50 |
| 4 flats, | $4\frac{1}{2}'' \times \frac{1}{2}''$, | 9.00 |
| | | 38.32 |

This area is about 32 per cent in excess. The only way in which it could be decreased without violating any item of the specifications would be to reduce the thickness of the angles and web to $\frac{5}{16}$ inch. An objection to the modified section is the difference between the area and thickness of the material in the cover and that in the angles and web. The chord might be narrowed an inch, but not to such an extent that ¶ 70 would allow the substitution of a $\frac{3}{8}$-inch cover; but this narrowing would doubtless require additional material in the end post, whose width must be the same as that of the upper chord. The section will therefore be left unchanged.

Using a horizontal axis of reference at the mid depth of the web, and adding algebraically the moments of the sectional areas of the cover and flats, the result is as follows:

| | | |
|---|---|---|
| Cover plate, | $9.62 \times 7.219$ | $= 69.45$ |
| Flats, | $9.00 \times 7.5$ | $= 67.50$ |
| | | $1.95$ |

The center of gravity of the section is therefore $1.95 \div 38.32 = 0.05$ inch above the center of the web. The moment of inertia may now be computed with reference to the neutral axis $AB$ in Fig. 39.

| | | | |
|---|---|---|---|
| Cover plate, | $\frac{1}{12} \times 22 \times (\frac{7}{16})^3$ | $=$ | $0.2$ |
| | $9.62\,(6.95 + 0.22)^2$ | $=$ | $494.6$ |
| Angles, | $4 \times 1.80$ | $=$ | $7.2$ |
| | $9.20 \times (7.00 - 0.83)^2$ | $=$ | $350.2$ |
| Webs, | $2 \times \frac{1}{12} \times \frac{3}{8} \times 14^3$ | $=$ | $171.5$ |
| | $10.50 \times 0.05^2$ | $=$ | $0.0$ |
| Flats, | $\frac{1}{12} \times 9 \times 1^3$ | $=$ | $0.7$ |
| | $9.00 \times (7.05 + 0.5)^2$ | $=$ | $512.9$ |
| | | $I =$ | $1537$ inches$^4$. |

The radius of gyration is

$$r = \sqrt{\frac{1537}{38.32}} = 6.33,$$

and the revised area 28.38 square inches.

Using the same value of $r$ the area required for $BC$ is $(\frac{1}{2} \times 61\,400 + 134\,900) \div 6650 = 24.91$ square inches. The section will be made the same as for $CD$ for the same reasons as those given in regard to reducing the section of $CD$.

The unit stress due to the weight of the upper-chord members is less than 10 per cent of the allowed unit stress and hence need not be taken into account in proportioning them (¶ 41).

In trusses with parallel chords and longer spans the section of the upper chord is varied in different panels in the interest of economy, the change being made in the webs and sometimes also in the angles by gradually increasing their thickness toward the middle of the truss.

The width of the chord section must always be such that its moment of inertia with reference to the axis *CD* (Fig. 39) shall not be less than that about the axis *AB*.

### ART. 56. SECTION OF THE END POST.

(See paragraphs 33, 38, 41, 70 and 90 of the Specifications in Art. 48.)

The upper-chord sections having a considerable excess of area, the section of the inclined end post will probably not differ much from it. Using the same value of $r$ the allowable unit stress for wind load is (¶ 33)

$$10\,500 - 60\frac{426.4}{6.33} = 6460 \text{ pounds per square inch.}$$

It is necessary to reduce the other stresses to equivalent wind-load stresses since the bending moment in the post is caused by the wind and has to be taken into account in determining its section. The stresses due to the wind exceed 25 per cent of the sum of the dead- and live-load stresses (¶ 38) and therefore the above unit stress is to be increased by 25 per cent. The equivalent wind stress is

$$\tfrac{3}{4}(57\,600) + \tfrac{3}{2}(132\,100) + 19\,700 + 18\,600 = 279\,600 \text{ pounds,}$$

and the area, $279\,600 \div 8070 = 34.65$ square inches. This area being less than the sectional area adopted for the upper chord the same composition will answer for the end post so far as resistance to bending in the plane of the truss is concerned.

The unit stress due to its own weight is found to be less than 10 per cent of the allowed unit stress, so that it will not need to be considered (¶ 41).

The end post forms a part of the portal bracing, and the wind pressure normal to the plane of the truss causes both end posts to bend in the plane passing through their center lines. It will be necessary therefore to compute the unit stress in the outer fiber in the post when considered simply as a column under the given direct stresses, and also that due to the bending moment, and compare their sum with the maximum allowable unit stress. If it exceeds the allowable value the sectional area must be increased until the sum of these unit stresses shall fall within the given limit.

The moment of inertia with reference to the axis $CD$ in Fig. 39 is now computed, the horizontal distance between the backs of the angles and $CD$ being 7.38 inches.

| | | |
|---|---|---|
| Cover plate, | $\frac{1}{12} \times \frac{7}{16} \times \overline{22}^3$ | $= 388.2$ |
| 4 angles, | $4 \times 2.66$ | $= 10.6$ |
| | $9.20(7.38 + 1.08)^2$ | $= 658.5$ |
| 2 webs, | $2 \times \frac{1}{12} \times 14 \times (\frac{3}{8})^3$ | $= 0.1$ |
| | $10.50(7.38 - 0.19)^2$ | $= 542.8$ |
| 2 flats, | $2 \times \frac{1}{12} \times 1 \times \overline{4.5}^3$ | $= 15.2$ |
| | $9.00(7.38 + 2.0)^2$ | $= 791.8$ |
| | | $I_1 = 2407$ inches$^4$. |

The radius of gyration with reference to the same axis is

$$r = \sqrt{\frac{2407}{38.32}} = 7.93.$$

In Art. 51 the bending moment was computed to be 51 030 pound-feet, and the distance from the neutral axis to the outer fiber (at the flats) is $7.38 + 2.0 + 2.25 = 11.63$ inches, hence the unit stress is

$$S = \frac{Mc}{I - \frac{Pl^2}{6E}} = \frac{51\,030 \times 12 \times 11.63}{2407 - \frac{279\,600 \times 276 \times 276}{6 \times 26\,000\,000}} = 3136 \text{ lbs. per sq. in.}$$

In this formula the denominator is $6E$ instead of $10E$ as in Art. 43, since the end post corresponds to one half of a beam whose ends are fixed and which is subject to a concentrated load at the middle. The half span is the clear distance from the portal bracket to the floor beam, and is estimated to be 276 inches.

While the quantity 10 500 in the column formula used above is not strictly the allowable stress in the outer fiber, yet it may be so regarded for columns in which the ratio of $l \div r$ is not very large. Substituting the value of $r$, and calling the stress in the outer fiber $S_1$ when the post is treated as a column only,

$$S_1 - 60 \frac{426.4}{7.93} = \frac{279\,600}{38.32},$$

whence $S_1 = 10\,530$ pounds per square inch, and the total unit stress is $S + S_1 = 3136 + 10\,530 = 13\,666$ pounds per square inch. The allowable stress is 25 per cent more than 10 500 or 13 125 pounds per square inch (¶ 38). The section therefore needs a slight increase.

Let the angles be increased in thickness to $\frac{7}{16}$ inch. This makes the area of the section 39.72 square inches, $I_1 = 2512$, $r = 7.95$, $S = 2997$, $S_1 = 10\,260$, and $S + S_1 = 13\,257$, which being so near the limit of 13 125 pounds per square inch may satisfy the requirements. If the formula $S = \frac{Mc}{I}$ had been employed the value of $S$ would have been 2835 instead of 2997 pounds per square inch.

## ART. 57. LATERAL AND TRANSVERSE BRACING.

(See paragraphs 30, 31, 33, 34, 37, 41, 96, 98 and 99 of the Specifications in Art. 48.)

The upper lateral ties will consist of bars of square section. The wind stress in the end tie is 9500 pounds, and the unit stress 15 000 (¶ 30), requiring an area of only 0.63 square inch, which being below the allowable limit (¶ 96), the size of $\frac{7}{8}$ inch square will be chosen (Pocket Companion, page 203). For the same reason all the upper lateral ties will be made of the same size.

The lateral struts will be composed of four angles united by vertical latticing, the upper angles extended across and riveted to the cover plates of the chords, and the lower angles resting on top of the horizontal leg of the lower angles of the chords. The lateral ties will also be attached to the cover plates, and hence the direct compression in the struts must be taken by the upper angles. The thickness of the latticing depends on the bearing strength of the rivets needed to transmit the vertical shear. The shear is 5880 pounds, and for a single system of lattice bars inclined 30 degrees with the vertical the stress in each bar is $5880 \times 1.155 = 6790$ pounds (Pocket Companion, page 244). As $\frac{3}{4}$-inch rivets will be used in the latticing this stress requires two rivets in a $\frac{3}{8}$-inch plate, the bearing strength of one rivet in a $\frac{3}{8}$-inch plate being 3380 pounds. The radius of gyration for two angles $3\frac{1}{2} \times 2\frac{1}{2}$ inches, with the longer legs horizontal and the backs separated $\frac{3}{4}$ inch, varies from 1.86 to 1.96 with reference to the vertical axis midway between them (Pocket Companion, page 152). Using the smaller value the allowable unit stress for the initial tension (¶¶ 33 and 34) is

$$9000 - 50\frac{192}{1.86} = 3840 \text{ pounds per square inch,}$$

and the area required is $8400 \div 3840 = 2.19$ square inches. Two angles $3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$ inch give an area of 2.88 square inches. For $3 \times 2\frac{1}{2}$ inch angles the area required would be 2.85 square inches, and two angles $3 \times 2\frac{1}{2} \times \frac{5}{16}$ inch give 3.24 square inches.

The bending moment at 4.5 feet from the center line of the truss is 43 660 pound-feet (Art. 51). The effective depth of the entire strut is 14.0 inches if the same angles be used above and below. Taking the unit tensile stress of 15 000 pounds (¶ 30) the net area in one flange is 2.50 square inches, and for the unit compressive stress of 10 500 pounds the gross area is 3.56 square inches. Two angles $3\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$ inch have a net area of $2(1.78 - 0.24) = 3.08$ square inches, a gross area of 3.56 square inches, and will be chosen for both flanges.

In view of the statements made in Art. 51 the direct compression due to wind and initial tension should not be combined with the stress due to flexure, but it may be of interest to see what the result would be if these stresses were regarded as simultaneous. It does not appear that the specifications contemplate that the full amount of initial tension referred to in ¶ 34 shall actually be put into the diagonal rods, but that the resulting stresses in the struts shall be used as a guard against designing the lateral struts without sufficient width for transverse strength. If, however, the compression in $CC'$ given in the table in Art. 51 be combined with the compression in the upper angles due to flexure and it be remembered that the wind stress in one diagonal of a panel displaces one half of its amount of initial tension in each rod of the same panel, it will add $8400 - \frac{1}{2}(3600) = 6600$ pounds to the stress and the additional area will be $6600 \div 10\,500 = 0.63$ square inch, making the total $3.56 + 0.63 = 4.19$ square inches. Two angles $3\frac{1}{2} \times 2\frac{1}{2} \times \frac{3}{8}$ inch would give 4.22 square inches.

Before proceeding further let the side elevation of the truss members be drawn whose sections have been designed, and also a section showing the elevation of an intermediate strut and a floor beam with the post connecting them. In order to put the floor beams as low as possible their lower flange angles should be clipped to a width of 3 inches, or sufficient to clear the main diagonal. The backs of the lower flange angles in the floor beams will thus be placed $7\frac{1}{2}$ inches higher than the center of the lower chord pins. After this the computation and drawing will progress together.

By drawing the bracket connecting the lateral strut and post the lever arm of its flange is found to be about 2.25 feet and its stress $23\,960 \div 2.25 = 10650$ pounds (Art. 51). The net section must be (¶ 37) $10650 \times 1.8 \div 15000 = 1.28$ square inches. The length of the flange is about 6 feet. For two angles $2\frac{1}{2} \times 2\frac{1}{2}$ inches and a $\frac{5}{16}$-inch web between, $r$ is at least 1.17 (Pocket Companion, page 151), since no angle thinner than $\frac{1}{4}$ inch is allowed (¶ 72). The unit stress is

$$10\,500 - 60\,\frac{72}{1.17} = 6810 \text{ pounds per square inch,}$$

and the gross area, $10650 \times 1.8 \div 6810 = 2.82$ square inches. The angles must therefore be $\frac{5}{16}$ inch thick, giving a gross area of 2.94 square inches, and a net area of $2(1.47 - 0.27) = 2.40$ square inches.

The lower laterals will be made of angles (¶ 24). As these can be stayed where they pass the lower flanges of the stringers the length to be used in the column formula is about 7.5 feet. The connection with the floor beams will be probably about a foot inside of the center of the chord. It will be assumed that in compression each lateral takes one half of the stress given in the table in Art. 51, its section to be designed in accordance with ¶ 37. The net section, however, will be made sufficient to take the entire tension given in the table. Pairs of angles with unequal legs are most suitable, as the radii of gyration are more nearly equal. The angles will be placed back to back and attached at intervals so as to act together as a column, and the longer legs will be placed horizontal and attached to the floor beams by connecting plates, which at present will be assumed as not less than $\frac{3}{8}$ inch thick. As many rivets will be needed to attach the angles in the first panel these angles will be taken wide enough to allow two rows of rivets. Applying the formula for wind stress in posts (¶ 33) the results are as follows:

| Member. | $r$ assumed. | Area req'd. Sq. in. | Composition and Section. |
|---|---|---|---|
| $ab'$ | 1.57 | 6.07 | 2 angles, $5'' \times 4'' \times \frac{3}{8}'' = 6.46$ sq. in. |
| $bc'$ | 1.08 | 5.01 | 2 angles, $3\frac{1}{2}'' \times 3'' \times \frac{7}{16}'' = 5.30$ |
| $cd'$ | 1.08 | 2.60 | 2 angles, $3\frac{1}{2}'' \times 3'' \times \frac{5}{16}'' = 3.86$ |

It is found that the horizontal legs of the angles have a net section exceeding that required to transmit the entire tensile stress, and hence the vertical legs of the angles need not be attached to the connecting plates (¶ 31) unless other reasons make this desirable.

The portal will consist of a small lattice girder of four panels with a double system of webbing, united by brackets with solid webs to the end posts (¶ 99). By drawing a side elevation of the end post and upper chord it is found that the clear head room required (¶ 4) will allow a depth (in the plane of the portal) over all of about 46 inches. Assuming that $4 \times 3$ inch angles may be used and taking the effective depth as the distance between the rivet lines in the flange angles, its value will be 42.5 inches. The maximum bending moment in the portal strut was computed in Art. 51 to be approximately 140 110 pound-feet, but from the side elevation of the truss and an outline diagram of the portal it is found that the point of inflection in the end post is 19 feet 8 inches from the pin at its upper end, and the center of moments for the maximum flange stress is in the center of the third panel of the portal strut. The revised moment is $M = 4437.5(19.67 - 0.67) - 19\,708 \times 9.75 = 107\,790$ pound-feet, and the flange stress is $107\,790 \times 12 \div 42.5 = 30\,440$ pounds.

The net section of the lower flanges is $30\,440 \div 15\,000 = 2.03$ square inches. As the flanges will be conveniently divided into four panels by the web the upper flange is unsupported in the plane of the portal for a distance of about 3 feet. This flange also has a uniform compression throughout of $2700 + 4200 - \frac{1}{3}(2700) = 5550$ pounds, making the total flange stress at the position of maximum moment $30\,440 + 5550 = 35\,990$ pounds. The unit stress in compression is

$$10\,500 - 60\frac{36}{0.88} = 8000 \text{ pounds per square inch,}$$

and the area $35\,990 \div 8000 = 4.50$ square inches. Two angles $4 \times 3 \times \frac{3}{8}$ inch (gross section $= 4.96$ square inches) will be used for both flanges, the longer legs being placed normal to the plane of the portal.

The vertical shear is 19 708 pounds, and as the braces make an angle of about 45 degrees with a line parallel to the axis of the end post the stress in each brace is $\frac{1}{2}(19\,708)1.414 = 13\,620$ pounds. As this requires five $\frac{3}{4}$-inch rivets in each end of the brace for a bearing of $\frac{3}{8}$ inch a wide angle will be used so as to allow two rows of rivets, thus reducing the size of the connecting plates. The smallest angles that permit two rows is $4\frac{1}{2} \times 3 \times \frac{3}{8}$ inch, which are also found to have ample strength when treated as columns.

The flange of the bracket has a lever arm of about 40 inches, with the center of moments about 2.4 feet below the upper pin in the end post. The flange stress holds in equilibrium the moment of the horizontal reaction of the end post, which may be considered as applied at the point of inflection. Its value is $4437.5(19.67 - 2.4)12 \div 40 = 22\,990$ pounds. Treating the flange as a column about 7 feet long similar to that of the intermediate bracket the area is found to be 5.04 square inches. Two angles $4\frac{1}{2} \times 3 \times \frac{3}{8}$ inch give 5.34 square inches and will also have an excess of net section. As, however, the end post is not free to rotate about the above center of moments the stress in the bracket flange will be less than the computed value. A slight reduction will accordingly be made in the size of the angles, two angles $4 \times 3 \times \frac{3}{8}$ inch being adopted.

## ART. 58. PIN PLATES.

(See paragraphs 40, 88 and 89 of the Specifications in Art. 48.)

The webs of members connecting with the pins must be reinforced by pin plates in order that the pressure on the projected semi-intrados of the pin holes may not exceed the limit specified in ¶ 40. The number of rivets in these plates is determined by the requirements of ¶¶ 39 and 88. The allowable bearing per linear inch for the $4\frac{1}{1}\frac{3}{8}$-inch pin is 57 700 pounds (¶ 40 and Pocket Companion, page 174). Since in many cases the rivets must be countersunk, no pin plate less than $\frac{3}{8}$ inch thick will ordinarily be taken (see Fig. 9). The allowable bearing of a plate (exceeding $\frac{3}{8}$ inch) for each $\frac{1}{16}$ inch of its thickness on a $4\frac{1}{1}\frac{3}{8}$-inch pin equals eight tenths of the shearing strength of a $\frac{7}{8}$-inch rivet, since

$$\frac{12\,000 \times 4.8125}{16 \times 4510} = 0.80.$$

If the stress is transmitted from or into a plate or angle $\frac{3}{8}$ inch thick the corresponding value is 0.916, and for a thickness of $\frac{5}{16}$ inch it is 1.10.

In order not to complicate this subject unduly for the student the clause in ¶ 40 reading "or one and a half times the allowed strain per square inch" will not be considered.

The linear bearing for each side of $ab$ is $\frac{1}{2} \times 151\,900 \div 57\,700 = 1.32$ inches, and for $bc$ is $\frac{1}{2} \times 191\,300 \div 57\,700 = 1.66$ inches. One of the pin plates on each side of the member must extend past the ends of the angles so as to engage enough rivets to transmit their stresses to the pin. Assuming the stresses to be divided in proportion to their areas the two main plates transmit 108 100 and the four angles 83 200 pounds. The number of rivets connecting one angle to the pin plate is $\frac{1}{4} \times 83\,200 \div 3940$ (bearing of $\frac{7}{8}$-inch rivet in $\frac{3}{8}$-inch angle) $=$ 5.3, hence 6 rivets will be inserted. If these two long pin plates are of the same thickness as the main plates they should be arranged to take one half of the stress after passing the angles. This requires that the number of rivets connecting each of them to a main plate equals $\frac{1}{4} \times 191\,300 \div 4510 =$ 11. This number is exceeded by the rivets already inserted. At $a$ the short outside pin plates must be $1.32-(2 \times 0.5)=0.32$

or $\frac{3}{8}$ inch thick. At $c$ two $\frac{3}{8}$-inch plates will also be added on the outside. This requires $6 \times 0.916$ or 6 rivets for each of these short plates, but if they transmit any tension past the pin a sufficient number of these rivets must be placed on the side toward $b$.

Assuming the stresses to be divided among the plates in proportion to thickness each of the four short pin plates at $c$ must carry past the pin a stress equal to $191\,300 \times 3 \div 28 = 20\,500$ pounds, which requires $20\,500 \div 3940 = 6$ rivets. Six rivets will therefore be placed on the left of the pin $c$ and four may conveniently be put on the right of the pin. The net depth of the plates is $12 - (4.81 + 0.03) = 7.16$ inches (¶ 78) and the net area of the main and the long pin plates combined is $7.16 \times 4 \times 0.5 = 14.32$ square inches, leaving a balance of $17.70 - 14.32 = 3.38$ square inches. If only the stress corresponding to this area were regarded as passing through the short pin plates, then only $\frac{1}{4} \times 36\,500 \div 3940 = 3$ rivets would be required. The net vertical section at the pin is $7.16 \times 2 \times 1.75 = 25.06$ square inches, or 145 per cent of the net section in the body of the chord $bc$. Some specifications call for only 125 per cent, while others demand 150 per cent. In order that the horizontal section behind the pin may have an area of 80 per cent of that of the member its length must be $17.32 \times 0.80 \div (2 \times 1.75) = 3.96$ inches. The chord will be extended 8 inches beyond the center of the pin, thus somewhat exceeding this requirement. A similar computation for the short pin plates at $a$ gives the same number and arrangement of rivets as at $c$. No rivet is put behind the pin, as that would aid the pin in splitting the plates; neither is any rivet put above nor below the pin, as that would reduce the net section. The strength of the member will be increased by placing the short pin plates with their fibers vertical.

The total linear bearing for the suspender $Bb$ is $63\,700 \div 57\,700 = 1.10$ inches, requiring only two plates, each $\frac{1}{2}(1.10) -$

0.32 = 0.23 inch thick. No first-class result could be secured by riveting a $\frac{1}{4}$-inch plate to a 0.32-inch web when the rivets must be countersunk, since the depth of the countersunk head of a $\frac{7}{8}$-inch rivet is $\frac{7}{16}$ inch (Art. 14). Two $\frac{3}{8}$-inch pin plates will be placed inside and two outside of the member. Dividing the stress in proportion to thickness, the two pin plates on each channel carry 22 300 pounds and require 22 300 ÷ 3360 = 7 rivets, 3360 pounds being the bearing value of a $\frac{7}{8}$-inch rivet in the 0.32-inch web. Eight rivets will be placed below the pin and two above it. The net cross section at the pin and the longitudinal section above the pin are very considerably in excess of that required.

The pressure of the pin against the lower part of the post *Cc* is equal to the vertical component of the stress in the main tie *Bc*, or 133 800 cos 41° 46′ = 99 800 pounds. The linear bearing for each jaw of the post is $\frac{1}{2}$ × 99 800 ÷ 57 700 = 0.87 inch and the pin plates must aggregate 0.87 — 0.45 = 0.42 inch in thickness. Plates $\frac{3}{8}$ inch thick will be placed both inside and outside. Their part of the total stress on the basis of thickness requires 7 rivets, while only 6 rivets would be needed for the bearing on the pin. Five rivets will be placed above the pin and two below it.

The bearing required at the upper end of the post *Cc* is less than that at the lower end as the total stress is only 57 400 pounds, but the same number and thickness of pin plates will be used. Only 4 rivets would be required by dividing the stress as before, while 6 rivets would be needed to develop the full bearing value of one $\frac{3}{8}$-inch pin plate. Paragraph 89 has some additional specifications to be met. The gross section of the four pin plates is 13.5 square inches, while that of the channels is 13.6 square inches. It is proposed to place the first rivets in the batten or stay plates 6 inches below the pin center, its upper extremity being $1\frac{1}{2}$ inches higher. The inner pin plates will therefore be extended for an additional line of rivets. It will be observed that the rivets in the pin plates are arranged so as not to come in the same cross sections as those in the stay plates.

While the stresses are smaller in *Dd* than in *Cc* the same pin plates will be used, as the sizes cannot well be reduced, for reasons already given, and uniform details will also decrease cost of manufacture. The pin plates on the upper chord at *C* transmit the horizontal component of the maximum stress in *Cd*, which is $69\,200 \sin 41° 46' = 46\,100$. Four $\frac{3}{8}$-inch plates will be used on the outside, two being filler plates and the other two placed on top of them and connecting with the flange rivets. The whole number of rivets on each side of the chord is $23\,100 \div 3940 = 6$, which is less than the smallest number that can practically be employed.

The required bearings of the upper chord and of the end post on the pin at *B* are 1.99 and 1.98 inches respectively, or practically 2 inches. Two of the outside pin plates of the chord and two of the inside pin plates of the end post are to be extended past the pin for the purpose of preventing any ordinary blow from displacing these members as well as to facilitate erection. These plates are called hinge plates or jaw plates. The other plates are faced parallel to the bisector of the angle between the axes of the members and at a distance of $\frac{1}{8}$ inch from it. In arranging the pin plates $\frac{1}{8}$ inch clearance is allowed between each hinge plate of one member and the nearest pin plate on the other. The webs are directly opposite, being of the same thickness, and the filler plates (between the angles) are made of the same thickness as the angles. Beginning at the outside the pin plates on the chord are $\frac{3}{8}$, $\frac{1}{2}$, $\frac{3}{8}$ and $\frac{3}{8}$ inch respectively, three being outside of the web; and on the end post $\frac{5}{16}$, $\frac{7}{16}$, $\frac{1}{2}$ and $\frac{3}{8}$ inch, two being outside of the web. One is allowed to be less than $\frac{3}{8}$ inch in order to equalize the aggregate thickness excluding the hinge plates, for otherwise the clearance of hinge plates would not be equal. The objection to this size is also less, as there is another plate before the web is reached.

Let the number of rivets in the pin plates of the chord be found. If the $\frac{3}{8}$-inch outside plate be the shortest, the $\frac{3}{8}$-inch inside and $\frac{1}{2}$-inch outside plates next, and the $\frac{3}{8}$-inch filler plate the longest, and if it were immaterial where the rivets were placed, the number of rivets in these successive plates would be 5.5, 11.0, 18.3 and 23.8, and the arrangement to meet this as nearly as possible is shown in Fig. 40. In obtaining these

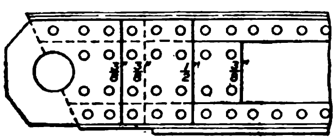


FIG. 40.

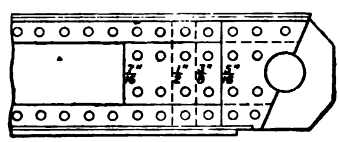


FIG. 41.

results it must be remembered that the stress in the plates must all be transferred either into angles $\frac{3}{8}$ inch thick or into a $\frac{3}{8}$-inch web, and hence the number of rivets depends on the bearing value of the rivets instead of their shear. The above arrangement is the one generally employed in practice, and frequently even a smaller number of plates connect with the flange rivets in proportion to the whole number used.

If the total stress be divided in proportion to the areas, the stresses and corresponding numbers of rivets in the parts composing the chord section are as follows:

| | Stress. | No. of Rivets. |
|---|---|---|
| $\frac{1}{2}$ cover plate, | 28 800 | 7.3 |
| 2 angles, | 27 500 | 7.0 |
| 1 web, | 31 400 | 8.0 |
| 2 flats, | 26 900 | 6.9 |
| | 114 600 | 29.2 |

As the web has its own bearing on the pin the total number of rivets required in the pin plates is $29.2 - 5.5 = 23.7$, which agrees with the total previously obtained. But the stresses in the cover plate and flats must be transferred from the pin plates through the flange rivets, hence the number of rivets connecting the pin plates to the upper flange angle is $7.3 + 3.5 = 10.8$; to the lower flange angle, $6.9 + 3.5 = 10.4$; and to the web, $8.0 - 5.5 = 2.5$. The arrangement of pin plates to conform to this requirement is shown on Plate IV. The two smaller sizes are placed with their fibers vertical to increase the factor of safety.

The arrangement of the pin plates and their rivets for the end post corresponding to that in Fig. 40 is shown in Fig. 41, the number of rivets required in the successive plates of increasing length being 5.5, 9.2, 16.1 and 22.1. Dividing the stress of 228 000 pounds between the several sectional areas the following results are obtained:

| | Stress. | No. of Rivets. |
|---|---|---|
| ½ cover plate, | 27 600 | 6.1 |
| 2 angles, | 30 400 | 6.8 |
| 1 web, | 30 100 | 7.7 |
| 2 flats, | 25 900 | 5.7 |
| | 114 000 | 26.3 |
| | | − 5.5 |
| | | 20.8 |

The number of rivets is computed on the basis of single shear for all except the web, and the final arrangement of plates and rivets is shown on Plate IV. The number of upper flange rivets connecting pin plates is $6.1 + 3.4 = 9.5$, and of lower flange rivets $5.7 + 3.4 = 9.1$. It will be observed that a few flange rivets are in double shear, so that their number would be reduced about 0.1, since the bearing on a $\frac{7}{16}$-inch plate is a little larger than the single shear of a rivet.

At the joint *a* the bearing required for the end post is about $2\frac{3}{16}$ inches on each side, as an additional vertical load of 19 400 pounds is transmitted by the end floor beam. This consists of a live load of 13 300, half a stringer and the track it carries, and half a floor beam, the position of the live load being that which produces the maximum stress in the end post. The plates which connect with the floor beam also form the pin plates placed directly inside of the web. The number of rivets required in the different parts, excluding three eighths of the plates inside of and adjacent to the web, is the same as for the upper extremity of the end post. The aggregate area of the forked ends of the end post at $a$ is 66.50 square inches (¶ 89). After passing the section 9 inches distant from the pin center the area rapidly increases and the forks are united on one side by the floor beam.

The maximum reaction of the leeward support is $(42\,950 + \frac{1}{2} \times 11\,450 + 14\,700) + (111\,820 + 16\,650) = 191\,800$ pounds, the last quantity in each parenthesis being the load transferred by the overturning effect of the wind. The linear bearing of one side of the shoe on the pin is therefore $\frac{1}{2} \times 191\,800 \div 57\,700 = 1.66$ inches. Two $\frac{1}{2}$-inch plates and two $\frac{3}{8}$-inch plates will be used on each side, one of the latter being a hinge plate.

It will now be necessary to see whether any modifications are needed in the pin plates of $ac$, since its sides must be parallel. Allowing a clearance of $\frac{1}{8}$ inch adjacent to shapes with countersunk rivets and $\frac{1}{16}$ inch between eyebars, the distance at $a$ from the center of the lower chord to the inside of the main plate of $ac$ is $7 + \frac{3}{8} + \frac{7}{16} + \frac{1}{8} + \frac{3}{8} + \frac{1}{8} + \frac{3}{8} = 8\frac{13}{16}$ inches, and at $c$ is $5 + \frac{3}{8} + \frac{1}{8} + 1\frac{7}{16} + \frac{1}{16} + 1\frac{1}{4} + \frac{1}{8} + \frac{3}{8} = 8\frac{3}{4}$ inches. This difference of $\frac{1}{16}$ inch may be added to the spacing at $c$, leaving the distance back to back of the angles of $ac$ $17\frac{5}{8}$ inches, or the inner pin plate at $c$ may be increased to $\frac{7}{16}$ inch. The former plan will be adopted.

On account of the provision in ¶ 86 the pitch of the rivets in the ends of the compression members must not exceed $3\frac{1}{2}$ inches. While on one hand it is desirable to reduce the length of pin plates by using the minimum pitch, on the other it is not desirable to have a smaller pitch than 3 inches; the rivets in all the pin plates will therefore be uniformly spaced 3 inches on pitch lines. It seemed preferable in this article to determine the number of rivets in the various parts of the structure by means of the computed stresses in the members, but it should be stated that usually the number of rivets is made large enough so as to develop the full strength of each plate or shape.

In computing the number of rivets in the pin plates of the upper chord and end post at $B$, the number is expressed to tenths in order that the student may more readily follow and check the computations, and also to show some equalities that otherwise might not be apparent.

Since this Article was written the results of some experiments on pin plates were published in a paper by T. H. JOHNSON read before the Engineering Society of Western Pennsylvania, reprinted from the Transactions in the Engineering Record, vol. 28, page 39, June 17, 1893. These indicate the importance of so designing the pin plates and their rivets that they may properly distribute the pressure of the pin to the plates and shapes composing the members.

## ART. 59. STAY PLATES AND LATTICING.

(See paragraphs 86, 87, 88 and 89 of the Specifications in Art 48.)

Compression members united by lattice bars have plates at the ends whose purpose is to aid in properly dividing the stress between the two segments of the member. These are called stay plates, or batten plates, or tie plates. The specifications require the stay plates at the upper end of the posts to be $1\frac{1}{2} \times 10 = 15$ inches long. The pitch of the rivets will be made 3 inches and the first rivets placed 6 inches below the center of the pin. For the middle post *Dd* it will be necessary to cut a strip out of the stay plate to clear the counter diagonal. Opposite the floor beam the channels in each post are united by a web and four angles, and as this web also performs the duties of a stay plate, those placed directly above it need not have the full length. The position and length of these stay plates will depend somewhat on the latticing. The pitch lines in the flanges of the channels are $1\frac{1}{2}$ inches from the backs, making the distance between the pitch lines of both channels $10 - 2(1.5) = 7$ inches. The pitch of the rivets in the latticing equals $2 \times 7 \tan 30° = 8.08$ or $8\frac{1}{16}$ inches, and $29\frac{1}{2}$ spaces will be required. If the stay plate is made 10 inches long its lower edge will be nearly on a level with the top of the floor beam. The pitch of rivets in this plate is $3\frac{1}{2}$ inches. The lattice bars will be $2\frac{1}{2} \times \frac{5}{16}$ inch in size (¶ 87), and the stay plates $\frac{3}{8}$ inch in thickness, the rivets being $\frac{3}{4}$ inch in diameter. In the suspender *Bb* the rivets in the pin plates and in the stay plates must not come in the same section, as that would reduce the net area of the channels below the limit used in designing their section.

The cover plate of the end post being 22 inches wide the stay plates will be $22 \times \frac{3}{8}$ inch and 2 feet 9 inches long. In order to clear the other members meeting at the same panel point the stay plate at each end must be placed 1 foot $6\frac{1}{2}$ inches from the section through the pin center. This will put the farthest rivets 4 feet 2 inches from the same section. For single latticing the bars must have a section of $4 \times \frac{3}{8}$ inch (¶ 87), but in order that the two $\frac{3}{4}$-inch rivets at each end of the bars will not have a smaller pitch than the minimum allowed it will require bars $4\frac{1}{2}$ inches wide. Adjacent bars will be placed so that the nearer rivets have a pitch of $3\frac{1}{2}$ inches. The pitch lines of rivets in the chord are $14 + 2(\frac{3}{8}) + 2(2) = 18\frac{3}{4}$ inches apart. Each pair of lattice bars will then require a space of

$2(18.75 \tan 30^\circ + 2.25 + 3\frac{1}{8}) = 32.40$ or $32\frac{3}{8}$ inches. The distance between the stay plates will admit 10 spaces, with a remainder of $\frac{7}{8}$ inch.

In the upper chord $BC$ the stay plates are of the same size as in the end post, their extremities being $12\frac{1}{8}$ and $10\frac{1}{8}$ inches respectively from the vertical sections through the pin centers. Six spaces of $32\frac{3}{8}$ inches each will just cover the distance between stay plates.

In the stiff chords $ab$ and $bc$ the pitch lines will be spaced $14\frac{1}{8}$ inches. The three stay plates are each $17 \times \frac{3}{8}$ inch and 2 feet long. As these are not compression members it will be convenient to place the adjacent ends of lattice bars side by side and with a slight clearance. If they were united by the same rivet it would separate the angles $\frac{3}{4}$ inch and require a filler on each side of the stay plates. If these adjacent rivets are spaced $2\frac{7}{8}$ inches the length of one panel of the latticing will be $2(2\frac{7}{8} + 14\frac{1}{8} \tan 30^\circ) = 22$ inches. Ten and one half panels of this length will fill the space between the stay plates, the first and last rivets being $2\frac{3}{4}$ inches from them. To facilitate shop work the continuous chord $ac$ is made symmetrical with respect to its center in all details except that the end at $c$ has two more small pin plates as previously stated.

The rivets may now be spaced in the members treated above. In the end post and upper chord the pitch is made $32\frac{3}{8} \div 7 = 4\frac{5}{8}$ inches, except at the ends, where the pitch of 3 inches is adopted. The specifications (¶ 86) require that the pitch shall not exceed $4 \times \frac{7}{8} = 3\frac{1}{2}$ inches for a distance equal to $2 \times 22 = 44$ inches. The 3-inch pitch is continued to the end of the stay plates. In the body of the chords $ab$ and $bc$ the rivets will be uniformly spaced $22 \div 4 = 5\frac{1}{2}$ inches (¶ 50), and so arranged that those in the horizontal legs of the angles shall stagger with the rivets in their vertical legs.

In the lateral struts the greatest stress is due to flexure and hence the end stay plate is not made as long as for a compression member. In Art. 57 it was found that the lattice bars must be $\frac{3}{8}$ inch thick and be connected with two $\frac{3}{4}$-inch rivets at each end. The strength in single shear is a little less than the bearing strength, but as the horizontal component of the stress in one bar is transmitted to the flange angles the shearing is really on more than one cross section of the rivet. The horizontal component of the axis of one lattice bar must not exceed $14 \tan 30° = 8.08$ inches, the pitch lines in the flanges being 14 inches apart. The panel length is taken slightly less than double this distance, or 16 inches.

## ART. 60. CONNECTIONS OF WIND BRACING.

(See paragraphs 31, 32, 59 and 97 of the Specifications in Art. 48.)

The upper lateral rods will be connected to the chords by means of clevises of standard form. Their smallest dimensions are for a rod $1\frac{1}{8}$ inches square as given in the Pocket Companion on page 166, and the corresponding diameter of the pin given is $1\frac{7}{8}$ inches. If the rod is smaller the clevises will allow a larger pin to be used. Let the finished diameter of the pin be $1\frac{15}{16}$ inches, then in order to develop the full strength of the rod used the thickness of the connecting pin plate must be $15\,000\,(\frac{7}{8})^2 \div 23\,250 = \frac{1}{2}$ inch (Pocket Companion, page 174). The plate must have $15\,000(\frac{7}{8})^2 \div 4510 = 3$ rivets connecting it to the cover plate and angle of the chord, but 5 rivets will be used as shown on the drawing. Where the connecting plate and splice cover of the chord come together as on the left side of $C$ the splice plate is extended to include the pin plate.

Instead of connecting the lateral ties with the cover plate of the chord two angles might be placed back to back and attached to its inner web, the pin being passed through the longer horizontal legs of the angles. This arrangement would answer only for small bridges, as it is objectionable to transmit large stresses into a web from its side. For long spans two rods are used for each diagonal of a panel, one being attached to the upper and the other to the lower part of the chord. In both of these cases both flanges of the lateral strut are designed on the assumption that the compression is divided equally between them. Each angle of the upper flange of the lateral struts is attached to the top of the chord by 3 rivets. The angles of the lower flange are fastened by only one rivet each, as the bracket supplies the rest of the necessary rigidity.

The intermediate bracket consists of a $\frac{5}{16}$-inch web plate with flange angles as determined in Art. 56, and connected to the lower flange angles of the strut by two angles $3\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$ inch, and to the post by two angles of the same size. The pitch of the rivets is made 4 inches throughout, $\frac{3}{4}$-inch rivets being used. In order to comply with the last sentence in ¶ 39 of the specifications a number of the rivets toward the ends of the bracket are replaced by turned bolts properly fitted (¶ 59). The radius of curvature of the flange angles is 6 inches at the ends and 4 feet for the intermediate portion. The extreme length of the bracket is 4 feet 3 inches both horizontally and vertically.

In designing the sections of the portal bracing the stress in each diagonal was found to be 13 620 pounds, which requires 5 rivets to unite it to the $\frac{3}{8}$-inch connecting plate. As the rivets are in single shear the bearing strength ought to be as nearly equal to the shearing strength as possible. The thickness of the connecting plates is therefore taken as $\frac{3}{8}$ inch. The difference of flange stress in adjacent panels is found either by chord increments or by the method of moments to be 19 708 pounds, which equals the shear in any section of the portal since the diagonals are inclined 45 degrees. The connecting plates and flange angles need 19 708 ÷ 3380 = 6 rivets to unite them. These rivets are given the minimum allowable pitch. By arranging all the rivets in the connection in rows both horizontally and vertically the connecting plates will have to be $10\frac{1}{2} \times 15$ inches in size.

The web of the bracket is made $\frac{3}{8}$ inch thick and extended upward through the lower flange of the strut and united not only to the flange angles but also to the web members. It is fastened to the middle of the inner web of the end post by two angles $3\frac{1}{2} \times 3 \times \frac{3}{8}$ inch, their longer legs being in contact with the web of the end post and about fill up the space between its flange angles. The radii of the edge of the bracket are 6 inches and 6 feet respectively, the sides of the bracket in position measuring 4 feet 6 inches and 6 feet, the latter measurement excluding the portion of the web in the side of the strut. The top flanges of the strut are extended over the upper chord of the truss and riveted to it by means of two plates, one a bent web plate to which the end diagonal is also riveted, and the other a bent cover plate with a filler under it of the same thickness as the connecting plate of the upper lateral rod.

The connecting plates for the lower laterals will be riveted to the bottom of the floor beam and passed between the angles composing the laterals, and as the rivets through the laterals are in double shear it is desirable to use plates $\frac{1}{2}$ inch thick in order to reduce the size of the plates by reducing the number of rivets required. The number of field rivets in each end of the laterals of the first panel is 47 500 ÷ $\frac{2}{3}$(5250) = 14; in those of the second panel 30 600 ÷ 3500 = 9; and in the third panel 15 900 ÷ 3500 = 5. The rivets used are $\frac{7}{8}$ inch in diameter. The number of rivets uniting the connecting plates to the floor beams, etc., at the panel points *a*, *b* and *c* depends on the strength of a rivet in single shear, the results being 16, 11 and 6. No connecting plates less than $\frac{7}{16}$ inch thick could be employed without increasing the last-named sets of rivets.

At the foot of the end post the connecting plate is not only riveted to the lower flange of the floor beam but also to an auxiliary horizontal angle which is riveted to the inner web of the end post by 5 rivets. The outer filler plate is modified on that side so as to extend to the upper edge of this angle. At the panel point *b* the connecting plate extends beneath the suspender to the outer edge of the stiff chord and is riveted by short angles to both of these members as well as to the floor beam.

The two angles forming each lateral are united at points about 2 feet apart throughout their length by rivets with half-inch washers $2\frac{1}{2}$ inches in diameter between the angles. This arrangement enables both angles to act together as one member.

The center lines of the laterals intersect in the central plane of the floor beam at points one foot inside of the center of the post and $7\frac{1}{4}$ inches higher than the center of the chords, and thus cause some flexure in the post and in the floor beam. These bending moments, however, are amply provided for, since at these places the members in question have an available excess of strength. There is no appreciable eccentricity in the connecting plates, since one half of the required number of rivets is always found on each side of the center line (produced if necessary), as may be seen by examining the drawing. The angles are connected by both legs and so aid in diminishing the area of the connecting plates.

The splice plate for one pair of laterals in a panel is shown in Fig. 35 (Art. 49). The laterals are riveted to the lower flanges of the stringers by means of bent plates passing between the angles of the former.

## ART. 61. PEDESTAL, EXPANSION ROLLERS AND BED PLATE.

(See paragraphs 26, 27, 103, 104, 107 and 108 of the Specifications in Art. 48.)

The pedestal is constructed by riveting the vertical pin plates which receive the entire reaction at a support to a horizontal bearing plate by means of four angles, $3\frac{1}{2}\times 3\frac{1}{2}\times \frac{5}{8}$ inch, two being inside and two outside. A thick angle is taken in order to aid in distributing the pressure over the bearing plate. The pin plates are finished so as to take direct bearing on the bearing plate, which at the expansion end rests on a set of friction rollers placed in a frame so as to maintain their spacing. The rollers move on a "rail plate" which itself is supported by the masonry.

The allowable pressure per linear inch for rollers 4 inches in diameter is $500\sqrt{4} = 1000$ pounds (¶ 104), and the maximum reaction of the leeward support at one end is 191 800 pounds (Art. 58). This requires six rollers each 32 inches long, which is satisfactory as it places the pin plates very nearly at the quarter points of the rollers. Allowing a clearance of $\frac{5}{8}$ inch between the rollers, and an additional space of one inch for the middle one of three connecting rods of the frame, the size of bearing plate will be 30 by 32 inches. If the bed plate were no larger than this it would reduce the pressure on the support to $191\,800 \div (30 \times 32) = 200$ pounds per square inch, which is far within the specified limit (¶ 103). The frame consists of two bars $5 \times \frac{1}{2}$ inch which receive the journals of the rollers and are bolted together by three parallel rods $\frac{7}{8}$ inch in diameter.

The rollers are supported on a series of 13 parallel rails riveted to a bed plate $\frac{3}{4}$ inch thick. The rails are of 50-pound standard section, with their heads brought to rectangular form by planing on the top and sides. One side of the base is removed almost entirely in order to space the rail head with a clearance of one half inch, the other side being fastened to the bed plate by six rivets. Such an arrangement allows the dust to drop down between the rails where it may be readily removed by a brush. So many friction rollers in bridges may be observed which become clogged with dirt in a short time (though this condition is partly due to lack of inspection) that a better arrangement than the usual one is very much needed. More detailed reference is made in Art. 20 to an expansion bearing in which the rails as used above form but one of several excellent features.

The bearing plate is made one inch thick, and to provide against lateral sliding the vertical bars are allowed to project $\frac{5}{16}$ inch above the surface of the rollers and the same or a larger distance below the top of the rails.

Four anchor bolts, $1\frac{1}{4}$ inches in diameter, will be used at each support (¶ 108). The maximum expansion of the truss is computed (¶ 27) to be $0.0000067 \times 142 \times 12 = 1.71$ inches.

## ART. 62. MINOR DETAILS, AND CAMBER.

(See paragraphs 4, 5, 62, 63, 91 and 109 of the Specifications in Art. 48.)

The upper chord will be spliced at the left of the panel points $C$ and $D$, and since the abutting ends will be planed the splices need only be sufficient to keep all the parts in contact (¶¶ 62, 63, 91). The stay plate is cut in two, one portion of which is also to act as a splice plate. The stay plate might be kept intact if the field rivets were placed on the other side of the splice than that usually adopted. The sizes of the plates and the arrangement of the rivets are shown on the drawing.

Although the backs of the angles of the upper chord and end post were regarded in Arts. 55 and 56 as flush with the web plates, which are 14 inches deep, their pitch lines would be so spaced in construction as to obviate any possible interference of the web with the cover plate and the flats, making the vertical distance out to out of angles $14\frac{1}{8}$ or $14\frac{1}{4}$ inches.

The riveting to transmit the maximum floor-beam reaction to the two channels of the post will now be considered. In Art. 50 it was found that 21 rivets would be required to connect the end angles of the floor beam to the inner side of the post in order to give sufficient shearing strength. The thinnest web of the posts is 0.32 inches and the bearing strength of a $\frac{7}{8}$-inch field rivet in this web is 2240 pounds. A $\frac{3}{8}$-inch diaphragm or web plate is united to both channels by means of four $3\frac{1}{2} \times 3 \times \frac{3}{8}$ inch angles, and through this the outer channel is assumed to receive its half of the maximum floor-beam reaction. The number of rivets needed to transmit its share from the inner channel connection is $\frac{1}{2}(61\,430) \div 2240 = 14$, which is less than that required for shear. The number of rivets in the diaphragm of the post must be at least $\frac{1}{2}(61\,430) \div 3940 = 8$, while 12 are employed.

The maximum load carried by the end floor beam is about two thirds of that for the intermediate ones. The bending moment is 197 100 pound-feet and the net flange area required is 7.76 square inches. The flange angles and cover plates will therefore be made $\frac{3}{8}$ inch in thickness. The web will also have ample strength when reduced to the same thickness. Its web and one of its upper flange angles is extended past the end post and riveted to each of the large pin plates by means of one $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{2}$ inch angle. As the bearing value of a field rivet in a $\frac{3}{8}$-inch plate is $\frac{2}{3} \times 3940 = 2620$ pounds, the web of the floor beam must be united to these two connecting angles by $41\,800 \div 2620 = 16$ rivets. This number will be inserted in addition to the rivets in the flange angles. Both of the lower flange angles and the strip of the web between them have to be cut off inside of the end post in order to clear the webs and upper angles of the latter. One angle of the same section as the flange angles is riveted to the lower edge of the extended web in order to increase the stiffness of the connection. The regular cover plate of the end post is discontinued at the top of the large pin plates, and a triangular cover plate is attached to these pin plates by two angles $3\frac{1}{2} \times 3 \times \frac{1}{2}$ inch and is riveted to the upper flange of the floor beam. Its extension beyond the width of the end post adds materially to its efficiency in fixing the lower extremity of the end post so far as lateral flexure is concerned.

The value of the end floor beam is not confined to its influence in stiffening the end post and thus reducing its weight, but also in doing away with the separate supports for the end stringers, thus reducing the number of supports of the bridge on the masonry from eight to four. One end of each span is hence free to expand without overcoming any friction between sliding bearings.

The distance from the center of the lower chord to the inside of the inner eyebar at $c$ is $5 + \frac{3}{8} + \frac{1}{8} + 1\frac{7}{16} + \frac{1}{16} = 7$ inches, the usual clearances of $\frac{1}{8}''$ and $\frac{1}{16}''$ being made as stated in Art. 54. At the panel point $d$ the distance to the same bar is $5 + \frac{3}{8} + \frac{1}{8} + 1 + \frac{1}{16} + 1 + \frac{1}{16} + 1\frac{1}{4} + \frac{1}{16} = 8\frac{15}{16}$ inches, while that to the inner eyebar in the third panel is $1\frac{5}{16}$ inches less. The divergence of these eyebars from parallelism with the axis of the chord is $1\frac{15}{16}$ and $\frac{5}{8}$ inch respectively. These divergences in a length of 23 feet 8 inches are allowable without requiring the eyebar heads to be bent so as to become truly parallel. The maximum divergence allowed is about $\frac{1}{8}$ inch to the foot.

The lower end of the suspender is attached by means of three short angles $5 \times 3\frac{1}{2} \times \frac{3}{8}$ inch to a horizontal plate which also serves to connect the lateral braces, this in turn being united to the flats of the lower chord by means of two other angles the rivets in whose vertical legs should stagger with those in the flats or plates in order not to reduce the net section of the chord. The greatest difference between the stresses in *ab* and *bc* due to the lower laterals is 39 400 pounds, and this amount must be transmitted through both sets of rivets of the angles connecting the plate to the lower chord. This stress equals the strength of 9 shop rivets and of 13 field rivets in single shear, and in order that the bearing strength of the rivets may not be less than the shearing the thickness of these angles must be $\frac{7}{16}$ inch.

The counter tie *Dc* in the main truss is made adjustable by inserting near one end a sleeve nut of standard form (Pocket Companion, page 216).

The broken lines inside of the projection of the portal show that the required clearance is obtained (¶¶ 4, 5). The specification does not state what allowance is made for the brackets, but several well-known specifications require the clearance of 14 feet wide to extend to a height of 15 feet, and a clear width of 6 feet at a height of 20 feet above the base of the rail.

The manner in which the camber in the truss is to be secured is specified in ¶ 109. This makes the distance between centers of the upper-chord pins equal to 23 feet 8 inches $+ (23.67 \div 80)$ inches $= 23$ feet $8\frac{9}{32}$ inches, when expressed to the nearest thirty-second of an inch. This will be the actual length of *CD*, but to obtain the length of *BC* one half of the clearance of the pin in the pin hole (¶ 78) must be added, making its length 23 feet $8\frac{5}{16}$ inches. The actual panel lengths of the lower chord are less than the nominal length by only the amount of the pin clearance. If the centers of the upper and lower chord pins be regarded as lying in concentric circles 26.5 feet apart, the length of a diagonal is found to be 35 feet $6\frac{7}{16}$ inches, after subtracting the full pin clearance of $\frac{1}{32}$ inch.

Another method of computing the actual length of the truss members is based on the specification that such a camber shall be provided that when the maximum live load is on the bridge the lower chord pins may be in a horizontal plane. This requires the determination of the simultaneous stresses due to dead and live loads in all the members when the live load produces the maximum moment at the middle of the truss, and that the strains due to these stresses together with the pin clearance shall be properly combined with the nominal lengths. For example, the stress in the chord *cd* for this loading is $61\,400 + 134\,400 = 195\,800$ pounds, the corresponding elongation is $(195\,800 \times 284) \div (25.00 \times 26\,000\,000) = 0.0856$ inch, and the actual length center to center of pin holes is $284 - 0.0856 - 0.0312 = 283.8832$ inches $= 23$ feet $7\frac{7}{8}$ inches, the coefficient of elasticity being taken at 26 000 000 pounds. The lengths of all the members are as follows:

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| *BC*...... | 23 feet | $8\frac{3}{32}$ | inches. | *Bb*...... | 26 feet | $5\frac{29}{32}$ | inches. |
| *CD*...... | 23 " | $8\frac{1}{16}$ | " | *Bc*...... | 35 " | $6\frac{7}{32}$ | " |
| *ab* = *bc*.. | 23 " | $7\frac{29}{32}$ | " | *Cc*....... | 26 " | $6\frac{3}{32}$ | " |
| *cd*...... | 23 " | $7\frac{7}{8}$ | " | *Cd*...... | 35 " | $6\frac{1}{4}$ | " |
| *aB*...... | 35 " | $6\frac{15}{32}$ | " | *Dd*...... | 26 " | $6\frac{3}{8}$ | " |

Sometimes it is desired that under full load there shall still be a slight camber. In this case the dead-load stresses may be increased by a certain ratio before being added to the live-load stresses, and the strains due to the combined stresses computed.

## ART. 63. FINAL ESTIMATE OF WEIGHT.

After computing the weight of each member and its details the results for one truss may be classified as follows:

| Truss Members. | Pounds. | One Half Lateral and Transverse Bracing. | Pounds. |
|---|---|---|---|
| Intermediate posts.. | 5 356 | Upper lateral ties.... | 518 |
| Suspenders........ | 3 010 | Upper lateral struts.. | 954 |
| Main ties.......... | 5 804 | Intermediate brackets | 756 |

| Truss Members. | Pounds. |
|---|---|
| Counter ties | 870 |
| Lower chord | 13 070 |
| Upper chord | 15 176 |
| End posts | 11 978 |
| Pins (steel) | 1 376 |
| | 56 640 |
| Pedestals | 1 422 |
| | 58 062 |

| One Half Lateral and Transverse Bracing. | Pounds. |
|---|---|
| Portal struts | 1 338 |
| Portal brackets | 806 |
| Lower laterals | 4 048 |
| | 8 420 |

The total weight of the above members exclusive of the pedestals is made up of the following items:

| | Pounds. | Per cent. |
|---|---|---|
| Main shapes and plates composing members. | 49 926 | 76.7 |
| Pin plates | 3 442 | 5.3 |
| Stay plates and latticing | 4 646 | 7.1 |
| Connections and other details | 3 954 | 6.1 |
| Rivet heads | 3 092 | 4.8 |
| | 65 060 | 100.0 |

The weight of the bridge, exclusive of pedestals and bearings, is divided as follows:

| | Pounds. | Per cent. |
|---|---|---|
| Trusses and wind bracing | 130 120 | 51.9 |
| Iron floor system | 64 095 | 25.5 |
| Track (as specified in ¶ 23) | 56 800 | 22.6 |
| | 251 015 | 100.0 |

The iron floor system is made up of two end floor beams each weighing 2957 pounds including its connections, five intermediate floor beams and twelve stringers whose weights are given in Arts. 49 and 50. The dead load on the end pin of each truss is $251\,015 \div 4 = 62\,754$ pounds, and that on the rollers is $62\,754 + 711 = 63\,465$ pounds.

If rails weighing 60 pounds per yard be used and the timber be southern yellow pine estimated to weigh $3\frac{1}{3}$ pounds per foot, board measure, the weight of the track (Art. 49) is found to be 298 or say 300 pounds per linear foot. If this value be substituted above the percentages become 53.9, 26.5 and 19.6 respectively, the weight of the bridge being 241,500 pounds. The weight of the floor system is $64\,095 \div 142 = 451$ pounds per linear foot, and that of the trusses and wind bracing is $130\,120 \div 142 = 916$ pounds per linear foot.

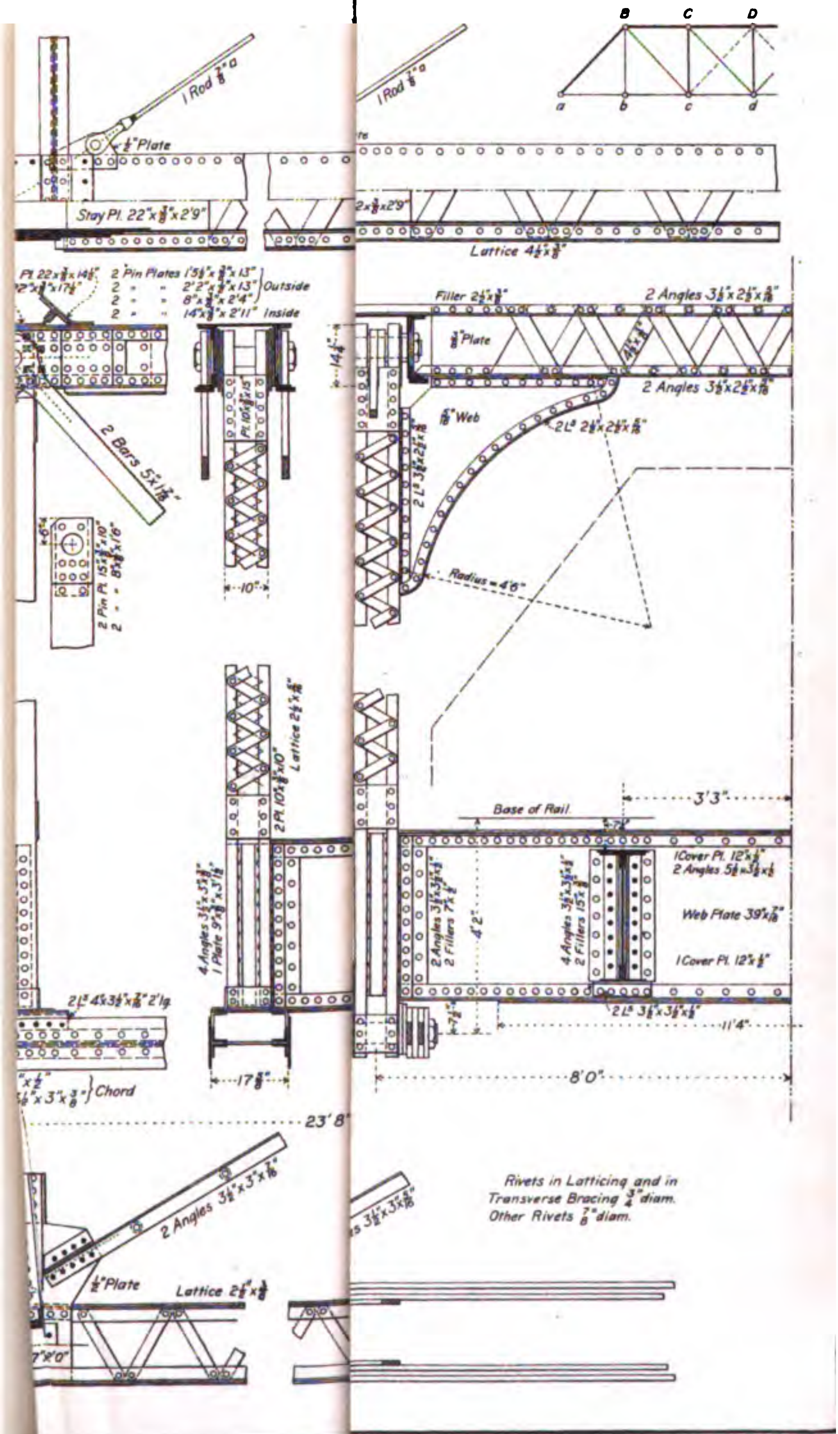
The apex load at *a* is computed to be 10454 pounds, whence the dead panel load for the truss is $(125\,508 - 20\,908) \div 5 = 20\,920$ pounds when the specified weight of track is included, and 19970 pounds when the actual weight of track is substituted. There is a considerable excess over the dead load assumed in Art. 51, as might be expected from the nature of the design. All the effects of wind were taken into account, thus increasing both main sections and details above those which would be obtained by following the customary methods in general practice.

## ART. 64. CONCLUDING REMARKS.

Since the assumed dead load is less than that obtained from the estimate in the preceding Article the design should be revised after re-computing the dead-load stresses. Not many sections of truss members will need to be changed since some have a small excess and most of the details have enough strength to take the comparatively slight additional stresses. It will also be necessary to review the computations to see whether the assumed distances agree with the results obtained or whether they are on the side of safety. This revision will be left as an exercise for the student.

The excess in the area of the upper chord might be avoided and the requirement of ¶ 90 more fully met by omitting the

TH


B
C
D
a
b
c
d
1 Rod
½" Plate
Stay Pl. 22"x⅜"x2'9"
Lattice 4½"x⅜"
2 Pin Plates
Outside
Inside
Filler 2½"x⅜"
2 Angles 3½"x2½"x⁵⁄₁₆"
⅜" Plate
2 Angles 3½"x2½"x⁵⁄₁₆"
⅜" Web
2 Bars 5"x1⅛"
Radius=4'6"
2 Pin Pl.
10"
Lattice 2½"x⅜"
Base of Rail
3'3"
4 Angles
1 Plate
2 Angles
2 Fillers
4'2"
1 Cover Pl. 12"x½"
2 Angles 5½"x3½"x½"
Web Plate 39"x⁷⁄₁₆"
1 Cover Pl. 12"x½"
2L³ 4"x3½"x⁷⁄₁₆" 2' lg.
Chord
17⅝"
8'0"
11'4"
23'8"
2 Angles 3½"x3"x⁵⁄₁₆"
½" Plate
Lattice 2½"x⅜"
7"x0"
Rivets in Latticing and in
Transverse Bracing ¾" diam.
Other Rivets ⅞" diam.

cover plate altogether and substituting stay plates and lattice bars like those on the lower side. The section for *CD* would then be composed of two web plates $14 \times \frac{9}{16}$ inch and four angles $3\frac{1}{2} \times 3 \times \frac{5}{8}$ inch, giving a total area of 30.64 square inches, a moment of inertia of 810.7 inches$^4$, a radius of gyration of 5.14 inches, and therefore a required sectional area of 29.76 square inches. The widths of the angles are here taken the same as those in the end post, although otherwise it would be preferable to use angles $4 \times 3\frac{1}{2} \times \frac{1}{2}$ inches, or to reduce the web to $\frac{1}{2}$ inch and employ angles $4 \times 3\frac{1}{2} \times \frac{9}{16}$ inch. The required area for *BC* being less than that for *CD* its web might be reduced to $\frac{7}{16}$ inch, the angles remaining the same. This arrangement would reduce the sectional areas a little more than 20 per cent and would thereby decrease the weight of the entire upper chord about 1535 pounds, exclusive of the reduction in pin plates, or a little more than 10 per cent. A serious objection to an upper chord without a cover plate is that it is more liable to deformation in transportation. This objection might be reduced by introducing transverse bracing within the chord at given intervals.

An economical post section is formed by four Z bars riveted to a single central web. If the web be placed perpendicular to the plane of the truss the moment of inertia of the post may be increased as required by the bending moment at the bracket by simply widening the web and so using but very little additional material. If this web be discontinued at the floor beam a very simple connection may be made by extending the floor-beam web between the Z bars and thus doing away with the usual connecting or hitch angles. This detail is embodied in the design of a Standard Short-span Through Bridge published in The Engineering Record, Vol. XXVIII, page 296, Oct. 7, 1893.

The student is also referred to the general drawing of the Madison Street Bridge of the Chicago and South Side Rapid

Transit Railroad in The Railroad Gazette, Vol. XXV, pages 651 and 652, Sept. 1, 1893. It shows one method of attaching the floor beams to the posts below the lower-chord pins, and a stiff lower lateral system having independent chords. Two papers on bridge design were read before the Engineers' Society of Western Pennsylvania in 1891; one by H. J. LEWIS entitled Bridge Design is reprinted from the Transactions of the Society in condensed form in Engineering News, Vol. XXVI, page 367, Oct. 17, 1891, and the other on Bridge Details by E. SWENSSON, abstracted in The Railroad Gazette, Vol. XXIV, page 156, Feb. 26, 1892. For an example of a long-span simple truss bridge see the paper of WILLIAM H. BURR on the Channel Spans of the Cincinnati and Covington Elevated Railway, Transfer and Bridge Company in the Transactions American Society of Civil Engineers, Vol. XXIII, pages 47–94, Aug. 1890, as well as the monographs referred to in Art. 6. The indexes named in that Article will guide the student to other examples of pin bridges found in the various engineering periodicals.

# CHAPTER VII.

## DESIGN OF A RIVETED TRUSS BRIDGE.

### ART. 65. DATA AND SPECIFICATIONS.

IT is required to design a highway through bridge for a small city with a roadway of at least 18 feet in the clear, one side being devoted to the usual highway transportation and the other to an electric street railway. The foot passengers are to be accommodated by a single sidewalk $5\frac{1}{2}$ feet in the clear.

The span is 80 feet between centers of bed plates and the structure is to be designed in accordance with WADDELL'S General Specifications for Highway Bridges of Iron and Steel, Second Edition. Those portions of the specifications to which reference is made hereafter in this chapter are reprinted in this Article by the consent of the author. For convenience of reference these paragraphs are numbered consecutively without regard to those omitted.

Although the span slightly exceeds 75 feet (see paragraphs 20, 21, 36 and 37 of the specifications) the trusses will be made of the triangular riveted kind. In order to stay the upper chord laterally—in the absence of an upper lateral system—the floor beams will be riveted to vertical members which are attached to the upper chord at its panel points. This type is known as the Warren truss with sub-verticals. The number of panels in one half of the truss is shown in Fig.

42. The floor beams are placed at *b, d, f,* etc., and are therefore spaced 16 feet. The effective depth of the trusses will be taken $9\frac{1}{2}$ feet or a little more than one ninth of the span (¶ 38). This will make the angle between a diagonal and the vertical nearly 40 degrees. The metal to be employed is wrought iron (¶ 22). The lateral system will be of the double Warren type, the braces meeting the chords midway betwee the floor beams and crossing each other at the centers of the floor beams.

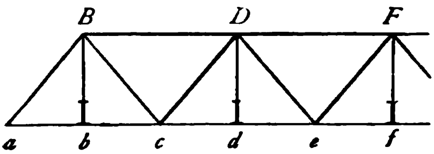


FIG. 42.

The following are the extracts from WADDELL'S Specifications applicable to this design.

1. Highway bridges shall be divided into four classes; viz., Class A, which includes those that are subjected to the *continued* application of heavy loads; Class B, which includes those that are subjected to the *occasional* application of heavy loads; Class C, those for ordinary light traffic; and Class D, those for unusually light traffic. In general it may be stated that bridges of Class A are for large, densely-populated cities; those of Class B for smaller cities and manufacturing districts; those of Class C for ordinary country roads; and those of Class D for districts where, on account of the hilly character of the country, large loads are unlikely to be transported, or for localities where the inhabitants are absolutely too poor to pay for heavier structures. In the latter case the bridges should be removed and replaced as soon as the traffic and resources warrant it. No bridge of Class D should have a span exceeding one hundred and sixty feet in length.

2. All bridges shall be proportioned to sustain the stresses produced by the following loads: 1st. The specified live load or loads. 2d. The dead load, which comprises the weights of all materials in the structure, and, in certain cases, an allowance for extraneous loading. 3d. The specified wind pressure, including both direct and indirect effects. 4th. Variation in temperature, when this affects the stresses, as for instance in a braced arch.

3. The live loads for main roadways of bridges of the different classes are to be taken from the following table:

| Span in Feet. | Moving Load per Square Foot of Floor. | | | |
|---|---|---|---|---|
| | Class A. | Class B. | Class C. | Class D. |
| 0 to 50....... | 100 | 100 | 80 | 65 |
| 50 to 100....... | 100 | 95 | 80 | 60 |
| 100 to 150....... | 95 | 90 | 75 | 55 |
| 150 to 175....... | 90 | 85 | 75 | 50 |
| 175 to 200....... | 85 | 80 | 70 | .. |
| 200 to 225....... | 80 | 75 | 65 | .. |
| 225 to 250....... | 75 | 70 | 60 | .. |
| 250 to 275....... | 70 | 65 | 55 | .. |
| 275 to 300....... | 65 | 60 | 50 | .. |
| 300 to 350....... | 65 | 55 | 45 | .. |
| 350 and over.... | 65 | 55 | 40 | .. |

Provided, however, that in no case shall the total live load per linear foot be less than the following: Class A, 1800 pounds; Class B, 1400 pounds; Class C, 1000 pounds; and Class D, 800 pounds.

4. When bridges have exterior sidewalks, the truss loads for the sidewalks are to be taken at four fifths (⅘) of those given in the last table. All live loads per square foot shall be applied to the width *in clear* specified for main roadway and sidewalks.

5. The live loads for joists, floor beams, cantilever brackets, beam hangers and primary truss members are to be one hundred (100) pounds per square foot of floor for bridges of classes A and B, eighty (80) pounds per square foot of floor for bridges of class C, and sixty-five (65) pounds per square foot of floor for bridges of class D, irrespective of the length of span.

6. When the main roadway of a bridge exceeds twenty (20) feet in width, the live loads thereon for floor beams and trusses may be reduced one (1) per cent for each foot of width exceeding twenty (20) up to a limit of forty (40) feet width, after which the reduction shall remain constant at twenty (20) per cent. It must be understood that this reduction does not apply to the joists.

7. This last rule is to be applied to the middle truss of a three-truss bridge by considering the width of roadway to be the clear perpendicular distance between outer trusses. It is to be applied also to trusses of bridges with sidewalks by considering the width of roadway to be the sum of the clear widths of main roadway and sidewalks.

8. In the case of bridges with exterior sidewalks, one sidewalk only and the roadway are to be considered loaded when proportioning the beam hangers and *primary* truss members of all bridges, and when proportioning the main truss members of all spans less than one hundred (100) feet for bridges of class A, and of all spans less than eighty (80) feet for bridges of classes B and C. In all other cases both of the sidewalks and the roadway are to be considered loaded. The eccentric loading increases the live load per truss. But when a bridge has only one exterior sidewalk the effect of the eccentric loading is to be considered to act upon the whole of the nearer truss; and the sidewalk is to be considered empty when calculating the stresses in the farther truss. Floor beams of bridges with one or two exterior sidewalks are to be proportioned on the assumption that, first, the main roadway is loaded and the sidewalk or sidewalks are empty, and, second, that the main roadway is empty and the sidewalk or sidewalks are loaded, due account being taken of the effect of reversing stresses as hereafter specified.

9. In addition to the preceding loads, the floor, joists, floor beams, beam hangers and primary truss members are to be proportioned for the following concentrated loads, which are, however, supposed to occupy a whole panel length of the main roadway to the exclusion of the other live loads there.

CLASS A.—A road roller weighing thirty thousand (30 000) pounds, of which twelve thousand (12 000) pounds are concentrated upon the roller in front of the machine, and nine thousand (9000) pounds on each of the wheels at the rear, the distance between the central planes of these wheels being five (5) feet, and that between their axis and the axis of the roller eleven (11) feet. The width of the roller is to be four (4) feet, and that of each wheel one foot eight inches (1′ 8″).

CLASS B.—A concentrated load of sixteen thousand (16 000) pounds equally distributed upon two pairs of wheels, the axes of which are eight (8) feet apart, and the central planes of the wheels six (6) feet apart.

CLASS C.—A concentrated load of ten thousand (10 000) pounds distributed in the same manner as for Class B.

10. The maximum stresses due to all positions of the above live loads and those due to the dead load shall be taken to proportion all the parts of the structure, the sectional areas being increased whenever necessary to provide for wind stresses, as hereinafter specified.

11. The dead load is to include the weight of all the metal and wood in the structure, excepting that of those portions resting directly on the abutments, whose weights do not affect the stresses in the trusses; also, if necessary, an allowance for snow, mud, paving or any other unusual fixed load that may ever come on the bridge.

12. Creosoted lumber is to be assumed to weigh four and one half (4½) pounds, oak and other hard woods four and one quarter (4¼) pounds, yellow pine three and one quarter (3¼) pounds, and white pine and other soft woods two and three quarter (2¾) pounds per foot board measure.

13. A bar of wrought iron one square inch in cross-section and three (3) feet long is to be assumed to weigh ten (10) pounds. Steel is to be taken two (2) per cent heavier, and cast iron six (6) per cent lighter than wrought iron.

14. Should in any bridge of or below one hundred (100) feet span the calculated dead load differ more than five (5) per cent from that assumed, or in any bridge from one hundred (100) to two hundred (200) feet span more than four (4) per cent, or in any bridge from two hundred (200) to three hundred (300) feet span more than three (3) per cent, or in any bridge exceeding three hundred (300) feet span more than two (2) per cent, the calculations of stresses, etc., are to be made over with a new assumed dead load.

15. The wind pressure per square foot for proportioning the lateral systems and sway bracing is assumed to be thirty (30) pounds for spans of one hundred (100) feet and under, and to decrease uniformly to twenty-five (25) pounds at three hundred (300) feet spans. For greater spans it is to be taken equal to twenty-five (25) pounds.

16. For structures in unusually exposed situations these pressures are to be increased by twenty-five (25) per cent. The total area opposed to the wind is to be determined by adding together the area of the vertical projection of the floor, joists, guard rails and wooden hand railing (if there be any) including hub planks, twice the area of the vertical projection of one truss with the floor beam attached, if suspended, and twice the area of the vertical projection of the iron hand railing, if there be any.

17. The total wind pressure on the structure is to be divided between the upper and lower lateral systems of through and deck bridges by imagining a surface midway between the upper and lower chords, and assuming that all the wind pressure on the parts above this surface will

be carried by the upper lateral system, and all on the parts below this surface by the lower lateral system.

18. But in order to prevent undue vibration under live loads the assumed wind pressures per lineal foot of bridge shall never be less than the following:

For lateral system of the *unloaded* chords:

| | |
|---|---|
| Class A.................................. | 150 pounds. |
| Class B.................................. | 125 " |
| Classes C and D.......................... | 100 " |

For the lateral system of the *loaded* chords:

| | |
|---|---|
| Class A.................................. | 250 pounds. |
| Class B.................................. | 225 " |
| Classes C and D.......................... | 200 " |

19. All wind pressures upon lateral systems are to be treated as moving loads. . . .

20. The length of span is to be understood as the distance between centers of end pins for trusses, and between centers of bearing plates for plate and latticed girders.

21. In general, spans of and below twenty (20) feet are to consist of rolled beams or simply wooden joists; spans of from twenty (20) to forty (40) feet of riveted plate girders; spans of forty (40) to seventy-five (75) feet of triangular riveted girders; spans of seventy-five (75) to ninety (90) feet of pony trusses with floor beams riveted to posts, and spans exceeding ninety (90) feet of pin-connected through or deck truss bridges.

22. All parts of the structure shall be of wrought iron or steel, except the flooring, floor joists, wheel guards, and in certain cases the hand railings. The wooden joists may be replaced by iron or steel beams, and the wooden floor by buckled plates with concrete and paving thereon.

23. There shall be two classes of floor system, viz., Class I and Class II.

Class I, used only for very important structures, shall be as follows: Stringers or joists of iron or steel, spaced not more than three and a half ($3\frac{1}{2}$) feet apart from center to center, shall be used under the main roadway. These shall be capped with nailing pieces about five (5) inches thick, upon which shall be nailed the floor, consisting of one thickness of three (3) inch plank running transversely or diagonally to the bridge, and another thickness of two (2) or three (3) inches laid diagonally, and crossing the lower planks at an angle not less than forty-five (45) degrees. The latter may be used as a wearing floor or may be covered with a pavement. The top flooring shall not exceed eight (8) inches in width when used as a wearing floor. The sidewalk floor shall consist of two (2) by six (6) inch planks nailed on transverse joists, which are supported by stringers of iron or steel. These wooden joists shall not be spaced more than two (2) feet centers. The under flooring of main roadway shall be fastened to the nailing pieces by wrought spikes seven (7) inches long by three-eighths ($\frac{3}{8}$) inch square; the top flooring shall be fastened by fifty (50) penny nails; and the sidewalk flooring by forty (40) penny nails, all of the best quality. The nailing pieces shall be fastened to the stringers by bolts with heads countersunk into the wood, and the sidewalk joists shall be attached to the stringers in a manner that will prevent displacement by wind. On account of the wide spaces between stringers, each stringer is to be assumed to support the entire weight that the wheels can bring upon it, and as much of the roller load as can come on the distance between consecutive stringer centers.

CLASS II.—The wooden joists shall rest either on the transverse floor beams or, preferably, on wooden shims that are firmly bolted to the floor beams. They shall be placed not more than two (2) feet from center to center, and shall lap by each other so as to extend over the full width of the floor beam or wooden shim, and shall be separated half ($\frac{1}{2}$) an inch, so as to permit the circulation of air. When they rest on wooden shims they shall be spiked thereto at each end by one seven (7) inch spike, driven obliquely into the end of the joist.

24. In cases where the elevation of the floor surface is fixed, and where there is not sufficient clearance between the lowest part of the bridge and the ground or bed of the stream below to permit of the joists resting on floor beams, they may be allowed to abut thereon, and rest on angle-iron brackets. The joists in such a case are to be spiked to the brackets or bolted to angle lugs on the web of the beam.

25. The cross section of the joists will depend upon their length and loading; but no joist shall be less than three (3) inches in width or twelve (12) inches in depth.

26. The joists shall be proportioned—

1st. For uniformly distributed loads by the formula

$$W = \frac{bd^2}{cl^2},$$

where $W$ is the safe uniformly distributed load in pounds, $b$ the breadth of the joist in inches, $d$ the depth of same in inches, $l$ the length in feet, and $c = 0.008$ for pine and other soft woods, and 0.00575 for oak and other hard woods. This formula provides that the joists shall not deflect more than one four-hundred-and-eightieth ($\frac{1}{480}$) of their span under the greatest uniformly distributed loads, and thus checks injurious vibrations.

2d. For concentrated loads by the formula

$$M = \tfrac{1}{6} Rbd^2,$$

where $M$ is the maximum bending moment in inch-pounds upon a joist due to concentrated and dead loads, $R = 1200$ for pine and other soft woods, and 1500 for oak and other hard woods, and where $b$ and $d$ are respectively the breadth and depth of the joist in inches. But when the bending is due to the weight of a road roller, the values of $R$ may be increased to 1500 for pine and other soft woods, and to 1800 for oak and other hard woods. This formula provides that the wood upon the lower edge of the joist shall not be strained beyond the proper limit.

27. In calculating moments, the concentrated loads are to be assumed to be distributed as follows:

FOR CLASS A.—The load upon the front roller is to be equally divided between all the joists which it can cover or partially cover at the same time, and the load upon each rear wheel is to be equally divided between the two nearest joists.

FOR CLASSES B AND C.—The load upon each wheel is to be divided equally between two joists.

28. The floor planks for roadways, if of pine or other soft wood, shall be at least three (3) inches thick, and if of oak or other hard wood at least two and one half ($2\frac{1}{2}$) inches thick.

29. They shall not be more than ten (10) or less than eight (8) inches wide, and shall be laid with one quarter ($\frac{1}{4}$) inch openings. Each plank shall be spiked to each joist upon which it rests by two (2) seven (7) inch cut spikes, the holes for which shall be bored in order to avoid splitting the timber. The thickness of the floor planks for sidewalks may be made as small as two (2) inches, and the spaces between them may be increased, if desired, to three quarters ($\frac{3}{4}$) of an inch. Their width is not to exceed six (6) inches. For the main roadways of bridges of class A there is is to be an additional wearing floor at least two inches thick. Bridges of class B may or may not have this additional wearing floor. All plank shall be laid with the heart side down.

30. Whenever in either class of floor system a wearing floor is used, the lower planks must be planed on one side and sized to a uniform thickness, so as to insure a perfect bearing between upper and lower floors.

31. In both classes of floor there shall be a wheel guard of a scantling not less than four (4) inches by six (6) inches on each side of the roadway to prevent wheel hubs from striking the trusses.

32. It is to be laid on its flat and blocked up from the floor by shims at least one (1) foot long, six (6) inches wide and two (2) inches thick, spaced not more than seven (7) feet centers, each shim being spiked to the floor by four (4) four-and-a-half (4½) inch cut spikes. The guard rails are to be bolted to the floor through the center of each shim by a three-quarter (¾) inch bolt. When the guard rails are bolted to the wooden hand-rail posts, the bolt heads are to be countersunk into the wood, so as to make a flush surface on the inner face of the guard rail. The joints in the guard rail are to be lap joints, at least six (6) inches long, each located symmetrically over the middle of a shim. When a bridge is on a heavy grade, the inner upper corners of the guard rails are to be covered with angle iron fastened to the timber by countersunk screws spaced about eighteen (18) inches apart, so as to protect the guard rails from the injurious effects of using them instead of wheel brakes for heavily loaded wagons.

33. When iron hand railing is employed, it is to be of a firm, substantial pattern, pleasing to the eye, and rigidly attached to the trusses or floor beams. Both through and deck bridges are to be provided with a hand rail on each side, not less than three and a half (3½) feet high above the floor. In case there be any liability of a horse jumping over this railing, its height must be increased to five (5) feet. There must be a hand rail on the outside of each sidewalk.

34. All floor timbers, guards and railings shall extend over all piers and abutments and make suitable connection with the embankments at the ends of the structure. Cast-iron aprons or cover joints shall be provided at the ends of spans, if required. The floor of the sidewalks shall extend to and connect with the floor of the main roadway so as to leave no open space between them.

35. Should there be one or more street railroad tracks crossing the bridge, there must be directly under each rail a joist or stringer, properly proportioned to resist the effect of the total maximum load on the rail;

and the bending effect of the concentrated loads upon the floor beams must be duly considered.

36. Any of the modern styles of truss, approved of generally by American engineers, will be accepted; but preference will be given to those in which the end posts are inclined and the intermediate posts are vertical, and in which there is a single system of triangulation.

37. The trusses shall be such as to give rise to no ambiguity of importance in calculating the stresses; and in all parts shall be so designed that the stresses to which they may be subjected under all conditions of loading can be calculated with sufficient accuracy for all practical purposes.

38. When floor beams are riveted to posts, the greatest depth for pony trusses is to be twelve (12) feet, and when floor beams are suspended it is to be nine (9) feet. Wherever possible the top chords of pony trusses and through plate or lattice girders are to be stiffened at each floor beam by side braces, having a batter not less than four (4) inches to the foot.

39. When the use of side braces is impracticable, as in bridges with sidewalks, the floor beams must always be riveted to the posts; and the top chords, batter braces and posts should be made fifty (50) per cent wider than the usual minimum in through bridges. The sections of side braces are not to be less than the following:

$2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$ angles for lengths of 7 feet and under.
$2\frac{3}{4}'' \times 2\frac{3}{4}'' \times \frac{1}{4}''$ angles for lengths of 7 to 10 feet.
$3'' \times 3'' \times \frac{1}{4}''$ angles for lengths of 10 to 13 feet.
$3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{4}''$ angles for lengths exceeding 13 feet.

40. No rods having less than three fourths ($\frac{3}{4}$) of a square inch sectional area shall be used in a bridge. No channels less than six (6) inches in depth are to be used for upper chords, batter braces or posts, or less than five (5) inches in depth for other members. No flat bars less than one half ($\frac{1}{2}$) inch thick or less than two (2) inches wide are to be used for diagonals or chord bars; nor any iron less than one fourth ($\frac{1}{4}$) inch thick anywhere in a bridge, except for filling plates. No truss strut is to have a ratio of unsupported length to least diameter exceeding forty-five (45), nor any other strut a ratio exceeding fifty (50). . . .

41. All spans are to be provided with some means of expanding and contracting longitudinally with a variation in temperature of one hundred and fifty (150) degrees Fahrenheit.

42. Spans of over seventy-five (75) feet are to have at one end nests

of turned wrought-iron or steel friction rollers running between planed surfaces.

43. Every span of a bridge must be anchored at each end to the piers or abutments, care being taken not to interfere with the longitudinal motion of the trusses due to changes of temperature and loading.

44. At the roller end of every span the rollers must be so grooved and the shoe plates and bed plates so tongued as to provide against the tendency of the shoes to slide in a direction transverse to the span.

45. Plate girders and lattice girders require no camber, but all pin-connected structures must be given a camber by making the horizontal panel lengths of the top chord longer than those of the bottom chord, in the proportion of three sixteenths ($\frac{3}{16}$) of an inch for every ten (10) feet.

46. Members subjected to either tension or compression combined with bending must be proportioned to resist the combined effects of these stresses. Top chords subjected to transverse loading should be made as deep as economy of material will permit.

47. If the extreme fibre stress resulting from the weight only of any strut exceed ten (10) per cent of the allowed intensity of working stress, the section of the strut must be increased so that the combined intensities of direct stress and bending shall not exceed by more than ten (10) per cent the said allowed intensity of working stress.

48. The bending effect of wind pressure upon batter braces is to be combined with the indirect stress or transferred load stress due to wind pressure, and the dead-load stress only, the structure being supposed to be empty when subjected to the maximum wind pressure. And in this case the intensity of working stress for the combined dead load and wind stresses may exceed by (50) per cent that employed for the combined live and dead load stresses in bridges of class A, and by twenty-five (25) per cent for bridges of classes B, C and D, the ratio of length to radius of gyration being assumed equal to zero.

49. Similarly in proportioning bottom chords, the sectional area required is to be determined, first, for the combined live- and dead-load stresses, employing the usual intensity of working stress; and second, for the combined dead-load stress, direct wind stress and transferred load or indirect wind stress, the intensity of working stress to be employed being the last-mentioned intensity, increased by fifty (50) per cent for bridges of class A, and by twenty-five (25) per cent for bridges of classes B, C and D.

50. Whenever it is practicable, the lateral struts, lateral rods, portal

struts, portal rods, intermediate overhead struts and vibration rods must be so connected that all the center lines of members of trusses, lateral systems and sway bracing coming together at an apex shall have a common point of intersection. If such an arrangement be impracticable, provision must be made for the bending moments induced by the eccentric attachment. In such cases, however, it is permissible to strain the metal fifty (50) per cent more than usual for bridges of class A, and twenty-five per cent more than usual for bridges of classes B, C and D, provided that the induced stresses be due to wind pressure. The attachment of the lateral system to the chords shall be thoroughly efficient. If connected to suspended floor beams the latter shall be stayed against all motion.

51. With the exception of top chords and batter braces, sections of all main members of bridges and viaducts are, whenever practicable, to be made symmetrical about two principal planes at right angles to each other, and containing the axis of the member, and the pin holes in the same must be located symmetrically in respect to one or the other of these principal planes.

52. The intensities of working tensile stresses for iron and steel in the various members are to be as given in the following table, when the span does not exceed one hundred and fifty (150) feet.

| Members. | Iron. | | Steel. | |
|---|---|---|---|---|
| | Class A. | Classes B, C and D | Class A. | Classes B and C. |
| Lower chords (plates or shapes) net section | 8 000 | 10 000 | 11 500 | 13 500 |
| Middle-panel diagonals (plates or shapes) net section | 7 500 | 9 000 | 9 500 | 11 500 |
| Hip verticals (plates or shapes) net section | 7 500 | 9 000 | 9 000 | 11 000 |
| Angle lateral ties | 12 500 | 12 500 | 16 000 | 16 000 |
| Flanges of rolled beams | 10 000 | 12 000 | 12 500 | 15 000 |
| Flanges of built beams, net section | 10 000 | 12 000 | 12 500 | 15 000 |
| * * * | * | * | * | * |

53. The intensities for main diagonals between end diagonals and middle-panel diagonals or counters are to be interpolated directly according to their position. . . .

54. Angle irons subjected to direct tension or compression must be connected by both legs, or the section of one leg only will be considered as effective; and the rivets must be staggered.

55. In members subject to tensile stress full allowance shall be made for reduction of section by rivet holes, screw threads, etc.

56. For truss members of bridges of class A the intensities of working compressive stresses in pounds are to be found by the following table:

| Conditions. | Iron. | Steel. |
|---|---|---|
| Flat ends.................... | $9\,000-30\frac{l}{r}$ | $12\,000-45\frac{l}{r}$ |
| One flat and one pin end..... | $9\,000-35\frac{l}{r}$ | $12\,000-53\frac{l}{r}$ |
| Pin ends.................... | $9\,000-40\frac{l}{r}$ | $12\,000-60\frac{l}{r}$ |

In which $l$ = length of member in inches from center to center of connections, and $r$ = least radius of gyration of section of member also in inches.

57. For truss members of bridges of classes B, C and D the intensities of working compressive stresses are to be found by adding twenty-five (25) per cent to the intensities given by the above table.

58. For members of the lateral systems and sway bracing of bridges of any class, the intensities of working compressive stresses are to be found by adding forty (40) per cent to the intensities given in the above table.

59. In any portion of a bridge in which the stresses of tension and compression alternate frequently, the sectional area required is to be determined by dealing with the part thus affected, first for the calculated maximum tension, then for the calculated maximum compression, employing for intensities of working stresses the values of $p'$ in the following formula:

$$p' = p\left(1 - \frac{1}{2}\,\frac{\text{smaller stress}}{\text{larger stress}}\right),$$

where $p$ is the intensity that would be used were there no reversion of stress, and adopting the greater of the two areas thus found.

60. The intensities of working shearing and bearing stresses on pins and rivets and the working bending stresses on pins are to be taken from the following table:

| Stress. | Iron. | | | Steel. | | |
|---|---|---|---|---|---|---|
| | Trusses. | | Lateral System and Sway Bracing. | Trusses. | | Lateral System and Sway Bracing. |
| | Class A. | Classes B, C and D. | | Class A. | Classes B, C and D. | |
| Shearing... | 7 500 | 9 000 | 11 000 | 9 000 | 11 000 | 13 000 |
| Bearing... | 12 000 | 15 000 | 18 000 | 15 000 | 19 000 | 22 500 |
| Bending... | 15 000 | 18 750 | 22 500 | 20 000 | 25 000 | 30 000 |

61. The intensities of working bearing stresses are to be measured upon the projection of the semi-intrados upon a diametral plane.

62. For rivets driven in the field, provision will be made twenty-five (25) per cent in excess of the above requirements.

63. RIVETS MUST NOT BE USED IN DIRECT TENSION.

64. In three (3) panel and four (4) panel spans the hip verticals are always to be stiffened so as to resist compression, and in all bridges preference will be given to designs having struts for hip verticals.

65. The distance between centers of rivets for plates strained in compression shall not exceed sixteen (16) times the thickness of plate in line of stress, nor thirty-five (35) times the thickness of plate at right angles to the line of stress, except in the case of top plates for upper-chord or batter-brace sections, when it may be as great as forty (40) times the thickness.

66. But if portions of the plate be held between two other plates or angle legs, the unsupported width may be taken as the distance between edges of said plates or angle legs instead of between centers of rivet holes.

67. All plates and angles used in chord sections should, if practicable, be ordered the full length of the section.

68. In partial splices of chord sections no dependence is to be placed on abutting ends, and the splices must be made of the full strength of the parts spliced.

69. But in complete splices at the end of sections perfect abutting of ends is to be relied upon. . . .

70. No shoe plate resting on masonry is to have a less thickness than one half ($\frac{1}{2}$) of an inch, and no plate resting on rollers a thickness less than five eighths ($\frac{5}{8}$) of an inch. When twelve (12) inch channels are used for the batter braces, these dimensions must be increased to five eighths ($\frac{5}{8}$) of an inch and three quarters ($\frac{3}{4}$) of an inch respectively; and

when eighteen (18) inch channels are used, they must be increased to three quarters (¾) of an inch and seven eighths (⅞) of an inch respectively. Moreover, in all cases, both of the above-mentioned plates must be properly stiffened either by their attachments or by independent angle irons.

71. Every bearing upon masonry must be provided with either a bed plate or a roller plate well fastened to the masonry by at least two (2) bolts not less than (1) inch in diameter; but if the shoe plate be sufficiently large, it may act as a bed plate at the fixed end of the span. Bed plates must be of such dimensions that the greatest pressures on the masonry shall not exceed those given in the following table:

| Material. | Permissible Pressure per Square Inch. | Material. | Permissible Pressure per Square Inch. |
|---|---|---|---|
| Am. Nat. cement concrete | 100 pounds | Extra good sandstone.... | 200 pounds |
| Brickwork.............. | 130 | Ordinarily good limestone | 250 |
| Portland cement concrete. | 150 | Extra good limestone.... | 300 |
| Ordinarily good sandstone | 150 | Granite................ | 450 |

72. The tension flanges of built floor beams and plate girders are to be proportioned by the formula

$$A = \frac{M}{DT} + A',$$

where $A$ is the area of the flange, $A'$ the area lost therefrom by one or more rivet holes, $T$ the intensity of working tensile stress, $M$ the greatest bending moment on the beam, and $D$ the depth between centers of gravity of upper and lower flanges. No part of the web is to be assumed to resist bending, its office being simply to take up the shear.

73. The compression flanges of built floor beams and plate girders are to have sectional areas at least as great as those for the tension flanges.

74. The web plates shall not be subjected to a shearing stress of greater intensity than that given in the following table:

| Classification. | Iron. | Steel. |
|---|---|---|
| Class A.................. | 4 000 | 5 000 |
| Classes B, C and D........ | 5 000 | 6 000 |

75. All web plates must have stiffening angles over bearing points, and at points of local concentrated loading, ordinary joist bearings excepted, also at intervals equal to about one and a half ($1\frac{1}{2}$) times the depth of the girder when the ratio of depth of web to thickness of same exceeds eighty (80).

76. No intermediate stiffeners need be used with a leg over three (3) inches in width perpendicular to the web of the girder, and this dimension may be reduced to two and a half ($2\frac{1}{2}$) inches when practicable.

77. No fillers need be used under stiffeners, but the stiffeners may be crimped (i.e., offset), so as to fit the web plates and angles. The ordinary limit of greatest offset may be taken at three quarters ($\frac{3}{4}$) of an inch; but this may be increased to one (1) inch for extreme cases.

78. The webs of plate girders with cover plates shall be overrun by the flange angles by an amount not exceeding one quarter ($\frac{1}{4}$) of an inch for each flange, in order that the web and cover plates may not interfere in assembling the girder for riveting.

79. The compression flanges of beams and girders shall be stayed against transverse crippling when their length is more than thirty (30) times their width.

80. The flange plates of girders shall be limited in width so as not to extend beyond the outer line of rivets connecting them with the angles, more than five (5) inches or more than eight (8) times the thickness of the first plate. When two or more plates are used on the flanges they shall either be of equal thickness or decrease in thickness outward from the angles.

81. In plate girders with flange plates at least one half of the flange section shall be angles, or else the largest-sized angles procurable must be used.

82. In determining the rivet spacing in the flanges of floor beams the flanges are to be divided in portions of lengths about equal to the depth, the stresses in the flanges are to be found at each point of division, and there must be enough rivets connecting angles to web between any consecutive points of division to take up the difference between the stresses at the points, providing that the rivets be not spaced more closely than three (3) diameters and not more than six (6) inches apart.

83. Plate girders shall have preferably a depth not less than one tenth ($\frac{1}{10}$) of the span.

84. Rolled beams shall be proportioned by the formula

$$M = \frac{2RI}{d},$$

where $M$ is the moment in inch-pounds, $I$ the moment of inertia of the section in inch measurement, $d$ the extreme depth of girder in inches, and $R$ the proper intensity of working stress.

85. Rolled beams used as longitudinal girders shall have preferably a depth not less than one fifteenth ($\frac{1}{15}$) of the span.

86. The greatest allowable pressure upon expansion rollers shall be determined by the formula

$$P = aD,$$

where $P$ is the permissible pressure in pounds per lineal inch of roller, $D$ is the diameter of the roller in inches, and $a$ is a constant to be taken from the following table:

| Classification. | Iron. | Steel. |
|---|---|---|
| Class A.................... | 350 | 500 |
| Classes B, C and D.......... | 440 | 620 |

87. The least allowable diameters for expansion rollers are one and three quarter ($1\frac{3}{4}$) inches for bridges of class A, and one and a half ($1\frac{1}{2}$) inches for bridges of classes B, C and D. The spaces between rollers must be made as small as practicable.

88. Finally, as regards the proportioning of any structure, if cases should occur which are not covered by the preceding specifications, the following rule is to be adhered to: "Details must always be proportioned to resist every direct and indirect stress that may ever come upon them under any probable circumstances, without subjecting any portion of their material to a stress greater than the legitimate corresponding working stress."

89. The rivets used shall generally be five eighths ($\frac{5}{8}$), three quarters ($\frac{3}{4}$) and seven eighths ($\frac{7}{8}$) inch in diameter. None smaller than one half ($\frac{1}{2}$) inch in diameter shall be allowed.

90. The pitch of rivets in all classes of work shall never exceed six (6) inches, or sixteen (16) times the thinnest outside plate, nor be less than three (3) diameters of the rivet. At the ends of compression members it shall not exceed four (4) times the diameter of the rivets for a length equal to twice the width of the member.

91. When two or more thicknesses of plate are riveted together in compression members, the outer row of rivets shall not be more than three (3) diameters from the side edge of the plate.

92. No rivet-hole center shall be less than one and one half ($1\frac{1}{2}$) diameters from the edge of a plate, and whenever practicable this distance is to be increased to (2) diameters.

93. No rivet shall be countersunk in a plate less than three eighths ($\frac{3}{8}$) of an inch in thickness.

94. In the cases of flanges or chords of plate girders, if the angle iron used has a thickness of three eighths ($\frac{3}{8}$) of an inch or more, the angle must be so chosen that seven eighths ($\frac{7}{8}$) inch rivets shall invariably be used.

95. The pitch of rivets in the compression flanges or chords of stringers or iron joists shall be uniform throughout the entire length and equal to the minimum, i.e., equal to that used near the ends.

96. Field riveting must be reduced to a minimum. It must be done with a button sett; the heads of the rivets must be hemispherical, and no rough edges must be left.

97. All the rivets in splice or tension joints are to be symmetrically arranged so that each half of any tension member or splice plate shall have the same uncut area on each side of its center line.

98. No rivet, excepting those in shoe plates and roller or bed plates, is to have a less diameter than the thickness of the thickest plate through which it passes.

99. The effective diameter of the driven rivet in punched holes shall be assumed the same as its diameter before driving; but in making deductions for rivet holes in tension members the diameter of the holes shall be assumed one eighth ($\frac{1}{8}$) of an inch larger than the undriven rivet. The effective diameter of the driven rivet in reamed holes shall be assumed one sixteenth ($\frac{1}{16}$) of an inch larger than its diameter before driving, and in making deductions for rivet holes in tension members the same allowance shall be made.

100. In the effective area of riveted members, pin, bolt and rivet holes shall be counted out for tension, and bolt and pin holes shall be counted out for compression.

## ART. 66. FLOOR SYSTEM.

(See Paragraphs 5–12, 22–35, 52, 59, 72–85 and 94 of the Specifications in Art. 65.)

Let Class II of the floor systems specified (¶ 23) be adopted, the depth of floor being reduced by means of the arrangement indicated in ¶ 24. Using a single layer of $3\frac{1}{2}$-inch white-oak plank (¶¶ 28 and 29) and white-pine stringers, the weight of the wooden floor is found, according to experience, to weigh about 23 pounds per square foot. With the maximum spacing of 2 feet between centers (¶ 23), the live load of 100 pounds per square foot of floor (¶ 5) and an allowance of 20 pounds per square foot for snow and mud (¶ 11), the weight supported by one joist whose span is 16 feet is $(23 + 100 + 20)2 \times 16 = 4580$ pounds. Assuming a depth of 14 inches (¶ 25) the deflection formula in ¶ 26 gives for this uniform load

$$b = 4580 \times 0.008 \times \overline{16}^2 \div \overline{14}^3 = 3.4 \text{ inches.}$$

The concentrated live load in ¶ 9 causes a maximum bending moment of 9000 pound-feet in one joist (¶ 27), while the dead and snow loads give an additional bending moment of 2752 pound-feet. Using a depth of 14 inches, $(9000 + 2752)12 = 1200 \times b \times 14 \times 14 \div 6$, whence $b = 3.6$ inches (¶ 26). Joists 4 by 14 inches will therefore be used for the roadway. One white-pine joist weighs 205 pounds (¶ 12) and carries 476 pounds of plank, and about 42 spikes weighing 14 pounds, making a total of 695 pounds, or say 700 pounds. This equals about 22 pounds per square foot, or a little less than that assumed for the wooden floor.

On the sidewalk the spacing of the joists may conveniently be reduced to 20 inches, and hence if the depth is the same the breadth of the joists must be $20 \times 3.6 \div 24 = 3.0$ inches on the assumption that the dead and snow loads are reduced only in the same ratio as the spacing. The dead load is really less than this, since white-pine planking 2 inches thick (¶ 29) will be employed. The joists will be laid on top of the cantilever brackets and lapped by each other with space between them for ventilation (¶ 24). If each joist is 17 feet long its weight is 164 feet, and about 5 pounds of 40-penny nails are required to nail the plank to it. The plank supported by one joist weighs 147 pounds. It is assumed that the sidewalk will be cleared of snow and hence the weight of the latter is not taken into account.

Both in the sidewalk and in the roadway the joists should be stayed sidewise by a line of bridging at their mid-span. Scantling 2 × 4 inches spiked to the joists will answer for this purpose.

The live load selected for the street railway consists of a Pullman double-deck motor car weighing 25 880 pounds equally distributed on two pairs of wheels with axles 7 feet apart, followed by a trailer weighing 22 500 pounds, distributed on two trucks whose centers are 18 feet apart, each truck having its two axles spaced 4 feet 9½ inches between centers. The maximum capacity of the motor car is 130 and of the trailer 200 passengers. Estimating the average weight of the passengers at 130 pounds the total weight on each wheel is 10 700 pounds for the motor car and 6060 for the trailer. The distance from the rear wheels of the motor car to the front wheels of the trailer is 13.1 feet.

An iron stringer is to be placed under each rail of the track (¶ 35) whose position is shown on the cross section of the bridge on Plate V. To provide sufficient clearance between the car and the truss, the farther rail is placed 8½ feet from the center of the truss. It is seen that one half of the width of the roadway is devoted to the electric railway, since the body of the cars is 8 feet wide and extends about 1½ feet beyond the rail. One wooden joist is put between the iron stringers, and the remaining joists are placed as shown on the drawing.

The maximum moment in the stringers due to the live load is produced by the motor car and equals 52 250 pound-feet. On account of ¶ 85 the depth of the I beams to be used as stringers must be 15 inches. Let the lightest section (as given in one of the handbooks), weighing 50 pounds per foot, be tried. Assuming the width of floor supported by them to be 2 feet 9 inches, the weight of plank is $476 \times 2.75 \div 2.0 = 655$ pounds, and if it be supposed that on the track the snow or ice will not exceed one half the weight allowed for the rest of the roadway, the snow load for a stringer is $10 \times 2.75 \times 16 = 440$ pounds. The weight of the rail and its spikes may be taken at 62 pounds per yard or 351 pounds per 16 feet. The rail and plank are supported on a white-pine nailing piece 5 inches thick (¶ 23) and 6 inches wide, bolted to the upper flange of the stringer. Its weight is 110 pounds and that of the bolts is about 4 pounds more. The total uniform dead load on the stringer is therefore 1920 pounds and the snow load 440 pounds. The maximum bending moment is then $52\,250 + \frac{1}{8}(1920 + 440)16 = 56\,970$ pound-feet. This requires an I beam whose moment of inertia of cross section is (¶¶ 52 and 84)

$$I = 56\,970 \times 12 \times 7.5 \div 12\,000 = 427 \text{ inches}^4.$$

The shape referred to above has a moment of inertia of 522 and hence has ample strength. If a 12-inch I beam be tried it is found that a heavier beam is required.

The cantilever bracket will be composed of a triangular web plate with flanges consisting of two angles each, the bracket being attached to the sub-verticals of the truss and to the web of the floor beam by means of splice plates as shown on the drawing. Although iron $\frac{1}{4}$ inch thick is allowed (¶ 40), none less than $\frac{5}{16}$ inch will be employed. Placing the center of the inner joist 6 inches from the center line of truss, and spacing the joists 20 inches apart, the lapped ends being separated as required in ¶ 23, the bracket will be subject to four loads of 316 pounds, each distant 8, 28, 48 and 68 inches respectively from the center line of truss. The iron hand railing (¶ 33) will be about 73 inches distant. A continuous railing (Albree's style O) is selected which weighs 28 pounds per linear foot, and 3 pounds per foot additional for braces and fittings. This gives a concentrated load of 496 pounds for the railing. The live load on the bracket is 100 pounds per square foot (¶ 5). The sidewalk is 5.5 feet wide in the clear, and the center of gravity of its live load is about 38 inches from the truss.

The bending moment at the support due to all these loads (not including the weight of the bracket itself) is 418 640 pound-inches, and for an effective depth of 24 inches the net section of the upper flange must be 1.5 square inches (¶ 52). No angles narrower than $2\frac{1}{2}$ inches will be used in the bridge, and if $\frac{3}{4}$-inch rivets be used in the bracket flanges, two angles $2\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$ inch give a net section of 2.4 square inches (¶ 99). Angles of the same size will be used for the lower flange. The $\frac{5}{16}$-inch web is also found to have an excess of section beyond that required for the vertical shear at its support (¶ 74).

The joists are held in place by being bolted to clips of angle iron $6 \times 3\frac{1}{2} \times \frac{3}{8}$ inch, 6 inches long, each of them being attached to the bracket flange by two $\frac{3}{4}$ inch rivets. Including these clips and bolts the weight of the bracket is computed to be 260 pounds, making the total dead load 2020 pounds. The moment of the dead load with reference to the center line of truss is 93 600 pound-inches and hence its center of gravity is 46 inches distant.

The floor beam will now be designed. The outer joists of the roadway support a guard rail (¶¶ 31 and 32) which, if made of white oak, will weigh 150 pounds including its shims, spikes and bolts. The concentrated dead loads on the floor beam are 850, 700, 700, 700, 700, 1920, 700, 1920 and 850 pounds when taken in order from left to right. The snow loads are 640, 640, 640, 640, 640, 440, 320, 440 and 640 pounds respectively. The live load is 100 pounds per square foot of floor (¶ 5) and covers a width of 8.5 feet, extending to a line 2 feet from the inner rail of the track. The live loads concentrated by the five joists on the left of the track are 2000, 3200, 3200, 3200 and 2000 pounds. The maximum floor-beam reaction for each rail due to the weight of the loaded electric cars is 17 820 pounds. As the uniform live load covers only a part of the roadway the slight reduction allowed in ¶¶ 6 and 7 will not be made. This will also offset the weight of laterals supported at the middle of the floor beam and which will therefore not be considered.

When the sidewalk is empty and the live load and snow cover the roadway (¶ 8) the left reaction equals 30 115 pounds and the vertical shear passes through zero at the inner iron stringer. The bending moment in the floor beam due to this external loading is 186 900 pound-feet. The web of the floor beam will be taken 28 inches deep, or about one eighth of the span (¶ 83), and its weight is estimated at 2400 pounds. The bending moment due to its own weight at the same section is 5900, making a total of 192 800 pound-feet.

When the sidewalk is loaded and the roadway empty, the total load exclusive of the weight of the floor beam is 19 860 and the left reaction is 16 750 pounds. The moment diagram is shown in Fig. 43. The moment is zero at a section about 6 feet 4 inches from the left support. This point is so far from the section where the positive moment is a maximum under the former loading that the moment is reduced more proportionately than the corresponding unit stress after applying the formula in ¶ 59.

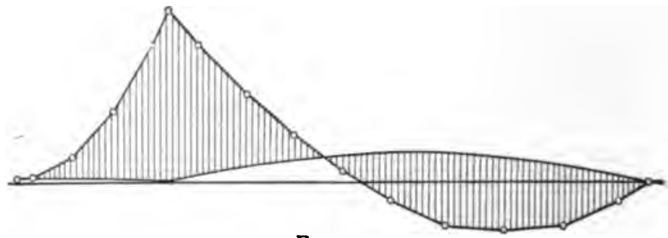


FIG. 43.

The concentrated load specified in ¶ 9 produces a smaller effect than those just considered, since the former excludes all other live loads from the whole panel.

For an effective depth of 25.1 inches the net area of the lower flange must be (¶¶ 72 and 52)

$$\frac{186\,900 \times 12}{25.1 \times 12\,000} = 7.68 \text{ square inches.}$$

Two angles $5 \times 5 \times \frac{1}{2}$ inch give a net area of 8.50 square inches (¶ 81) provided only one rivet hole is deducted from the section of each angle (¶¶ 55 and 99). The rivets are $\frac{7}{8}$ inch in diameter (¶94). The upper flange may have the same composition (¶ 73). The flange angles will be united to the web by two rows of rivets, and since the lateral braces are to be attached to the lower flange at the middle (Art. 65) it will be necessary to use there only a single row of rivets for a short distance in order to avoid the necessity of deducting two rivet holes for each angle and thus requiring a heavier one.

The greatest reaction under any loading occurs at the right support and equals 35 225 pounds. The web therefore requires a net section of $35\,225 \div 5000 = 7.05$ square inches (¶ 74). If not more than 9 rivet holes have to be deducted the web need not exceed $\frac{3}{8}$ inch in thickness. This is certain since fillers should be used under the end connecting angles, thus placing the rivets in two rows.

The minimum pitch of flange rivets (¶ 82) must also be found at the right end of the floor beam. The difference of flange stress between the support and the nearest joist 15 inches distant is 21 730 pounds, making the pitch for a single row 3.46 inches (¶¶ 60 and 61). As there are two rows of rivets they will be spaced 6 inches in each row. This arrangement provides a larger number of rivets toward the middle of the beam than is theoretically required.

As the ratio of the depth of web to its thickness is less than 80, no stiffeners are required (¶ 75) except at the location of the iron stringers. One $3\frac{1}{2} \times 3 \times \frac{1}{2}$ inch stiffener will be placed on one side of each I beam and riveted to it, the flanges being cut away on that side for this purpose. On the other side of each beam a short connecting angle of the same section will be used. The stiffener will be crimped over the flange angles (¶ 77). The maximum shear in the stringer due to the live load is 16 720 and that due to the dead and snow loads is 1180 pounds. As the web of the stringer is $\frac{1}{2}$ inch thick it will require $(17\,900 \div 6560)1.25 = 3.4$ or 4 field rivets (¶¶ 60–62). The number of rivets uniting the connecting angles to the web of the floor beam must not be less than $(1920 + 440 + 17\,820) \div 4920 = 4.1$ or 5 shop rivets, or 6 field rivets. The stiffeners will be riveted to the floor beam in the shop, while the other angles will be riveted in the field.

Each support for the joists consists of one horizontal angle $3\frac{1}{2} \times 3 \times \frac{3}{8}$ inch, two vertical angles $3 \times 3 \times \frac{5}{16}$ inch, and one filler $3 \times \frac{3}{8}$ inch, all of these shapes being 6 inches long. Four rivets unite the support to the web of the floor beam. Angle lugs $3\frac{1}{2} \times 3 \times \frac{1}{2}$ inch and $\frac{3}{4}$-inch bolts are used to hold the joists in position.

The weight of the floor beam may now be computed and it is found to be 2320 pounds. This includes the weight of the 14 three-quarter-inch bolts just referred to, but does not include the splice plates and their rivets at the ends of the floor beam, and which cannot be designed until the section of the lower chord of the truss is determined.

The end floor beams may have their flange angles reduced to a thickness of $\frac{5}{8}$ inch, the other dimensions remaining unchanged. This diminishes the weight 295 pounds.

## ART. 67. STRESSES.

(See Paragraphs 2–13, 15–20, of the Specifications in Art. 65.)

The floor-beam reaction at the truss adjacent to the sidewalk, due to the weight of the floor of the main roadway, but exclusive of that of the floor beam itself, is 4146 pounds. The reaction due to the sidewalk floor and bracket is 2410 pounds, and one half of the weight of one floor beam is 1160 pounds. The weight of the truss and one half of the lower lateral system is estimated at 250 pounds per linear foot or $250 \times 16 = 4000$ pounds per panel. The dead panel load is therefore $4146 + 2410 + 1160 + 4000 = 11\,716$, or say 11 720 pounds. The corresponding reaction due to the snow load is 2760 pounds.

The live load to be used in computing the stresses in the "main truss members" is not the same as that specified for the joists, floor beams and cantilever brackets. In this example the live load per square foot of floor is 95 pounds (¶ 3) and that on the sidewalk is four fifths of this amount, or 76 pounds (¶ 4). The load in the roadway covers only the portion nearer the sidewalk, as stated in the preceding Article in connection with the design of the floor beam. This gives a live panel load for the truss of 17 280 pounds (¶ 8). For the "primary truss members," or sub-verticals in this case, the live load is the same as for the floor beam (¶ 5), the panel load being 20 220 pounds. The dead, snow and live panel loads at the end floor-beam connections are three fourths of the value of the others.

The stresses due to these panel loads are given in the following table. That portion of the dead panel load which comes from the weight of the truss and laterals should really be divided, nearly one half of it being applied at the upper chord. This division, however, only affects the sub-verticals, and hence 1800 pounds is deducted from the tension which would exist in the sub-verticals *Dd* and *Ff* if the entire dead load were applied at the floor-beam connection.

TABLE OF STRESSES.

| Members. | Dead Load. | Snow Load. | Uniform Live Load. | Concentrated Wheel Loads. | Maximum. |
|---|---|---|---|---|---|
| *aB* | − 34 470 | − 8120 | − 50 830 | − 24 160 | − 117 580 |
| *Bc* = − *cD* | + 22 980 | + 5410 | + 35 580 | + 16 940 | + 80 910 |
| *De* = − *eF* | + 7660 | + 1800 | + 19 770 | + 9920 | + 39 150 |
| *Bb* | + 6990 | + 2070 | + 15 170 | + 10 140 | + 34 370 |
| *Dd* = *Ff* | + 9920 | + 2760 | + 20 220 | + 10 730 | + 43 630 |
| *BD* | − 37 010 | − 8720 | − 54 570 | − 23 210 | − 123 510 |
| *DF* | − 56 750 | − 13 360 | − 83 670 | − 35 700 | − 189 480 |
| *ac* | + 22 210 | + 5230 | + 32 740 | + 15 560 | + 75 740 |
| *ce* | + 51 810 | + 12 200 | + 76 400 | + 34 100 | + 174 510 |
| *ef* | + 61 680 | + 14 530 | + 90 950 | + 37 890 | + 205 050 |

The stresses due to the concentrated wheel loads were found graphically by the method explained in Art. 31, under the condition that the entire load was supported by one truss, but as the center of the track is 6 feet and ½ inch from the nearer truss this one receives 69.8 per cent, while the farther truss receives 30.2 per cent. Only the stresses in the latter truss are inserted in the table.

As it is supposed that the full live load will not be on the bridge when the wind pressure is a maximum the stresses due to wind need not be computed for any members except the laterals. Only two members in the half span of the truss are subject to reversal of stress, viz., *De* and *eF*. The live-load stresses in each of these members are 8470 pounds for the uniform and 4550 pounds for the concentrated loads, and hence the minimum stress is $8470 + 4550 - 7660 = 5360$ pounds. The stress due to the overturning effect of the wind reduces the magnitude of this minimum stress, but as its value will be comparatively very small it will not now be considered.

Using the wind pressure specified in ¶ 18, the wind panel load for each system of lateral braces is $\frac{1}{8}(125 + 225)16 = 2800$ pounds. As nearly this entire amount is applied at the floor beam, it will be on the safe side to regard the whole of it as so applied. At the end floor beam the panel load is reduced one fourth. This gives stresses of $\pm$ 8070, $\pm$ 5650 and $\pm$ 3140 pounds (¶ 19) in the laterals on the left of the first, second and third or middle floor beams respectively. The stress in the strut connecting the shoes is a compression of 6300 pounds minus the friction of the windward shoes, the latter being equal to 3660 pounds for a coefficient of friction of 0.15 and a release of weight due to the wind of 2000 pounds.

The above stresses refer to the truss adjacent to the sidewalk, all of the stresses in the other truss being somewhat less. If two trailers, instead of one, followed the motor on the street railway the stresses in the two trusses would have nearly the same magnitudes.

## ART. 68. SECTIONS OF TRUSS MEMBERS

(See Paragraphs 2, 40, 46–49, 51–59 and 64–66 of the Specifications in Art. 65.)

The minimum lower-chord section will be made up of a central vertical web and two angles, and this will be increased by the addition of one or more cover plates. In order that the increments of chord stresses may be distributed by the connecting plates as directly as possible into the shapes composing the chord, these plates will be placed between the web and the angles. This arrangement will require the insertion, beyond the connecting plates, of small fillers under the angles that add nothing to the net strength of the lower chord, but it is considered that the advantages secured more than compensate for this additional material. The section is shown in Fig. 44.

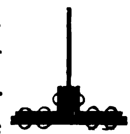


FIG. 44.

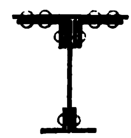


FIG. 45.

As the upper chord requires a wide section, angles will be used whose vertical leg is only sufficient to give ample room for a single row of rivets in splicing, and whose horizontal leg is wide. The lower part of the web also requires stiffening by means of two angles (¶¶ 65 and 66). The arrangement is shown in Fig. 45 (¶ 51). It will be desirable to use a web of the same dimensions as well as angles of the same widths in both chords.

The net sectional areas of the lower chord are 7.57 for *ac* (see Fig. 42), 17.45 for *ce*, and 20.50 square inches for *ef* (¶¶ 52, 55, 99 and 100). Its composition is as follows:

| | |
|---|---|
| 1 web, $14'' \times \frac{7}{16}''$, $6.12 - 2(0.44)$ ......... | 5.24 sq. in. |
| 2 angles, $6'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$, $2(3.97 - 0.44)$... | 7.06 |
| Net area of *ac* ................ | = 12.30 |
| 1 cover plate, $14'' \times \frac{3}{8}''$ .................. | 5.25 |
| Net area of *ce* ................ | = 17.55 |
| 1 cover plate, $14'' \times \frac{5}{16}''$ ................ | 4.38 |
| Net area of *ef* ....... ....... | = 21.93 |

There is considerable excess in the net section of *ac* which will allow more than two rivets to be placed in a vertical row if necessary. To reduce the size of the angles throughout and increase the first cover is not desirable since all the stress in the covers has to be transferred through the angles from the connecting plates at the joints, and to reduce them only in the end panel would require an additional splice. No thinner plate than $\frac{5}{16}$ inch will be used.

The above areas were obtained by deducting the rivet holes in the web and vertical legs of the angles. If a section be taken through the rivets in the horizontal legs of the angles and which stagger with the others, the net sections will be 14.06, 17.69 and 21.45 square inches respectively. The centers of gravity of the net chord sections are 3.51, 2.73 and 2.16 inches from the horizontal backs of the angles. The computation referred to in ¶ 49 applies to trusses with overhead bracing and a comparatively narrow roadway, in which case the lower chord in the end panel may have its stress reversed.

Without giving the preliminary computation for the upper chord its composition and gross sectional area are found to be as follows:

| | |
|---|---|
| 1 web, $14'' \times \frac{7}{16}''$ .................. | 6.12 sq. in. |
| 2 angles, $6'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ ............ | 6.84 |
| 2 fillers, $3\frac{1}{2}'' \times \frac{5}{16}''$ ............... | 2.18 |
| 2 angles, $3'' \times 3'' \times \frac{5}{16}''$ ............ | 3.56 |
| Area of $BD$ ............... | = 18.70 |
| 1 cover plate, $14'' \times \frac{3}{8}''$ ............ | 5.25 |
| Area of $DF$ .............. | = 23.95 |

The least moments of inertia of these sections occur with reference to a vertical axis through their respective centers of gravity (see Fig. 45) and equal 78.3 for $BD$ and 164.0 for $DF$, the radii of gyration being 2.05 and 2.62. If, in view of the manner in which the upper chord is stayed, the condition of the ends of the chord members be regarded as intermediate between flat ends and pin ends, the corresponding unit stresses are 7150 and 8040 pounds per square inch (¶¶ 56 and 57), and the required areas 17.28 and 23.68 square inches. The centers of gravity of these sections are 5.29 and 4.08 inches below the horizontal backs of the upper angles.

The chords may now be placed in position on the drawing. The lines of action of the stresses in truss members meeting at any point should intersect in one point. This is not practicable at every joint since the centers of gravity of the chord sections are located at different elevations, but the eccentricity should be reduced to a minimum. Let two horizontal lines be drawn at a distance apart equal to the depth used in computing the stresses, or 9.5 feet. The backs of the angles in the lower chord will be placed $2\frac{3}{4}$ inches below the lower line, and those in the upper chord $4\frac{1}{4}$ inches above the upper line. The latter measurement might have been $4\frac{1}{8}$ inches, but by increasing it to $4\frac{1}{4}$ inches the distance out to out of chord angles is made 10 feet 1 inch. This reduces the effective depth 1.8 inches for *ac* and 0.76 inch for *BD*, the effect of which, however, is covered by their excess in area.

The stress in the end post *aB* being nearly equal to that in *BD* its section will be made the same. As there is no upper lateral system and hence no portal bracing the investigation referred to in ¶ 48 need not be made in this case.

The length of a diagonal measured between the intersections of center lines at the joints is 149 inches. The condition of the ends of the diagonal struts will be taken the same as for the upper chord. The web members will be of four angles, two being placed on each side of the connecting plates, and so united at intervals as to act together. In this manner eccentric connections are avoided.

The strut *cD* consists of four angles, $4 \times 3 \times \frac{7}{16}$ inch, with a gross area of 11.48 square inches, a least radius of gyration of 1.58 inches, an allowable unit stress of 7120 pounds (¶¶ 56 and 57), and a required area of 11.36 square inches. The longer dimension is adjacent to the connecting plates. As the strut

$eF$ is subject to alternating stresses the allowable unit stress must be reduced by means of the formula in ¶ 59. The member is composed of four angles, $3 \times 3 \times \frac{5}{16}$ inch, with a gross area of 7.12 square inches, a least radius of gyration of 1.26 inches, an allowable unit stress of 5660 pounds, and a required area of 6.92 square inches.

The tie $Bc$ requires a net area of 8.09 square inches (¶ 52), and is formed of four angles, $4 \times 3 \times \frac{3}{8}$ inch, whose net area is 8.40 square inches. The minimum stress in $De$ is compression, whence its allowable unit stress is reduced from 9000 (¶¶ 52 and 53) to 8380 pounds (¶ 59). The net area required is 4.67 square inches, and that furnished by four angles $3 \times 3 \times \frac{5}{16}$ inch is 5.88 square inches.

If the angles in $Dd$ be taken $4 \times 3 \times \frac{5}{16}$ inch, the maximum pressure of the wind against the truss (¶ 48) produces a bending moment in $Dd$ at the top of the floor beam of 5250 pound-feet and together with the dead and snow loads causes a stress in the outer fiber fully equal to the limit allowed (¶ 52). This size would be manifestly too light, as no margin would remain to stay the upper chord (¶ 46). If the angles be taken $\frac{3}{8}$ inch thick they will be able to sustain, in addition to the maximum tension of 43630 pounds, a bending moment equal to that due to a force of about 450 pounds applied transversely at the center of gravity of the upper chord. A plate $9 \times \frac{3}{8}$ inch will be inserted between these angles to increase their transverse stiffness, and the angles must be crimped at the top of the floor beam in order to allow the splice plates and fillers on each side of the web to pass between them. Although the stress in $Bb$ is not as great as that in $Dd$ or $Ff$, it will be made of the same strength, thus having increased power to stay the upper chord at the hip joint.

The end diagonals of the lateral system have a maximum stress of 8070 pounds, either tension or compression. They are attached to the lower flange of the floor beam and hence are divided into two equal parts, each 154 inches long. The ends are considered as pin ends. Using $\frac{3}{8}$-inch connecting plates, these diagonals will consist of two angles, $3\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$ inch, with the longer legs horizontal and on opposite sides of the connecting plates, and have a least radius of gyration of 1.09 inches, an allowable unit stress of 2350 pounds (¶¶ 56, 58 and 59), a required area of 3.44 square inches and an actual area of 3.56 square inches. The net area must be 1.29 square inches (¶¶ 52 and 59), which is less than half that provided. Angles of the same size are needed for the laterals between the floor beams *b* and *d*. Two angles $3 \times 2 \times \frac{5}{16}$ inch would answer for the remaining laterals so far as strength is concerned, but since $\frac{3}{4}$-inch rivets will be employed and the angles are to be connected by both legs (¶ 54), two angles not less than $3 \times 2\frac{1}{2} \times \frac{5}{16}$ inch must be used. As these angles differ so little in weight from the $3\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$ inch angles and their use requires a change in the location of rivets in three sets of connecting plates the same size will be employed throughout. The student should notice that any single angle or pair of angles with a least radius of gyration less than 0.80 would reduce the allowable unit stress to zero. As the strut between the shoes has to sustain a compression of only 3340 pounds, one angle $3 \times 3 \times \frac{3}{8}$ inch will transmit it. It will be stayed at intervals by the end joists, and in turn will hold them in position laterally by means of angle clips and bolts and may be riveted to the outer face of the end posts by means of bent plates as shown on the drawing.

A series of experiments relating to the net section of a plate or shape having more than one line of rivets is described by B. B. FLINT in the Transactions of the American Society of Civil Engineers, Vol. XXVII, pp. 406–411, Oct. 1892.

## ART. 69. JOINT DETAILS.

(See Paragraphs 50, 54, 60, 61–63, 65, 68, 69, 90, 97, 99, 100, of the Specifications in Art. 65.)

The web members should now be inserted on the side elevation of the truss, their center lines having been previously drawn. A plan of the lower chord and laterals may also be commenced as the sections of the members shown on it are now known. After this the student will find it convenient to proceed with the computations and drawing together.

The strength of a $\frac{7}{8}$-inch shop rivet in single shear is 5410 pounds, while its bearing strength is shown in the following table (¶¶ 60, 61 and 99):

| | | | | | | |
|---|---|---|---|---|---|---|
| Thickness of plate | $\frac{5}{16}''$ | $\frac{3}{8}''$ | $\frac{7}{16}''$ | $\frac{1}{2}''$ | $\frac{9}{16}''$ | $\frac{5}{8}''$ |
| Bearing strength.. | 4100 | 4920 | 5740 | 6560 | 7380 | 8200 |

As stated in a preceding Article a $\frac{5}{16}$-inch connecting plate is placed on each side of the web of the chord to which the web members of the truss are riveted. The sub-verticals require $43\,630 \div 8200 = 6$ rivets at their upper ends, while $Bc$ and $cD$ need $80\,910 \div 8200 = 10$ rivets at each end. $De$ and $eF$ must have at least $39\,150 \div 8200 = 5$ rivets, but as these members have alternating stresses this number will be increased to seven.

At $B$ the web of the upper chord is extended past that of the end post and united to the angles of the same. The entire stress in $BD$ must be transferred from its upper angles and web to the connecting plates, as it is not presumed that the ends of the fillers and lower angles will abut against the connecting plates. Divided in proportion to area, the upper angles carry 58 300 and the web 65 200 pounds. Six rivets in the angles can transmit $6 \times 2 \times 4920 = 58\,040$ pounds to both web and plates and $6 \times 8200 = 49\,200$ pounds to the plates alone. The difference requires two additional rivets in order to be transferred back again from the web to the connecting plates. The stress in the web needs 65 200 ÷ 5740 = 12 rivets for its transmission. In the end post the stress to be received by the web from the connecting plates is 55 520 pounds, requiring 10 rivets, while the angles receive 62 060 pounds and therefore need 8 rivets.

The sum of the horizontal components of the maximum stresses in *cD* and *De* is somewhat larger than the maximum chord increment at *D*. Its value is 77 330 pounds, about two thirds of which should be transferred to the upper angles and the remainder to the web. Seven rivets in the former and five in the latter will answer the purpose.

The web of the upper chord will be spliced at *F*, the connecting plates of the joint also acting as splice plates. The stress to be thus transmitted (¶ 68) is 63 650 pounds, which requires 12 rivets to unite each end of the web to the connecting plates. It is not practicable to conform in this case with the requirement of ¶ 97 on account of the double duty of the splice plates.

At *a* the end post is extended to the bearing plate, the larger angles, web and connecting plates being finished to take bearing. The angles of the lower chord are extended until they meet the outer angles of the end post. Six rivets are required in the angles and the same number in the web to transmit the full chord stress into the connecting plates. As the last three rivets in the angles also bear on the web of the end post they can transmit three fourths of the angles' share of the stress, which amounts to 43 470 pounds.

At *c* it is proposed to splice the web of the chord by means of the connecting plates. The stress in the web of *ce* is 174 510 × 5.24 ÷ 17.55 = 52 080 pounds, and hence requires 10 rivets. If the adjacent section through the rivets in the horizontal legs of the angles were taken the stress would be 174 510 ×

$6.12 \div 17.68 = 60\,380$ pounds, requiring 11 rivets. The sum of the horizontal components of the maximum stresses in *Bc* and *cD* equals the maximum chord increment at *c*, its value being 104 130 pounds. In order to distribute this increment properly the connecting plates must have nine of its rivets to pass through the angles and seven through the web of *ce*. On the side toward the middle of the truss the connecting plates must be extended so as to have only two rivet holes taken out of the web of *ce* at the first section, otherwise the net section of the chord would be reduced below the required limit.

At *e* the maximum chord increment is very nearly one half of that at *c* and hence the corresponding numbers of rivets needed bear the same ratio to those at *c*.

The cover plates are in all cases extended to the end of the connecting plates in order that they may receive their stresses as directly as possible. By extending the outer cover a little farther at *e* it will obviate the necessity of extending the connecting plates at *c*.

The angles of both chords will be spliced (¶ 68) near the middle of the truss as shown on the drawing. One angle of the lower chord can take a tension of $3.53 \times 10\,000 = 35\,300$ pounds, which requires 7 rivets in single shear. The same number of rivets is required in a splice of the upper-chord angle. If $5 \times 3$ inch splice angles are employed their thickness must be $\frac{9}{16}$ inch for the lower and $\frac{1}{2}$ inch for the upper chord.

As the section of the lower chord of the truss has been determined the connection of the floor beam to the truss and to the sidewalk bracket can now be designed. The drawing shows that the splice plates may have a width of $9\frac{1}{2}$ inches, allowing three horizontal rows of rivets. Nine rivets can conveniently be put in one end of the plates without interfering with the nearest joist (¶ 97). Below the splice plates fillers will be used under the connecting angles, and these should have one row of rivets outside of the angles. This arrangement gives seven rivets connecting the end angles and fillers to the web of the floor beam in addition to those through the splice plates. Only six of the nine rivets in the splice plates can be considered as helping to take the vertical shear, since each rivet has a bearing in the connecting angle of ¾ inch or twice the thickness of the floor-beam web. This makes 13 rivets available for the vertical shear, provided the splice plates are also ⅜ inch thick.

If the lower flange angles be finished to bear against the lower-chord angles of the truss the lever arms of the rivets in the splice plates will be 13.5, 17.0 and 20.5 inches. If $S$ be the stress in one of the upper line of rivets, 20.5 inches from the center of moments, the resistance of the nine rivets will be (see Art. 37)

$$3 \times S \times \frac{\overline{13.5}^2 + \overline{17.0}^2 + \overline{20.5}^2}{20.5} = 428\,000 \text{ pound-inches.}$$

The second member of this equation is the moment of the cantilever and its loading with reference to the same center of moments in the center line of truss. The value of $S$ is found to be 3280 pounds. If the vertical shear due to the maximum loading in the roadway be regarded as uniformly distributed among the 13 available rivets each rivet must resist $28\,865 \div 13 = 2220$ pounds. The maximum stress therefore in any rivet is the resultant of 3280 and 2220 pounds, which is 3960 pounds. This is very nearly equal to the allowable pressure on a field rivet (¶ 62). This stress applies only to the upper line of rivets in the splice, the others being considerably less.

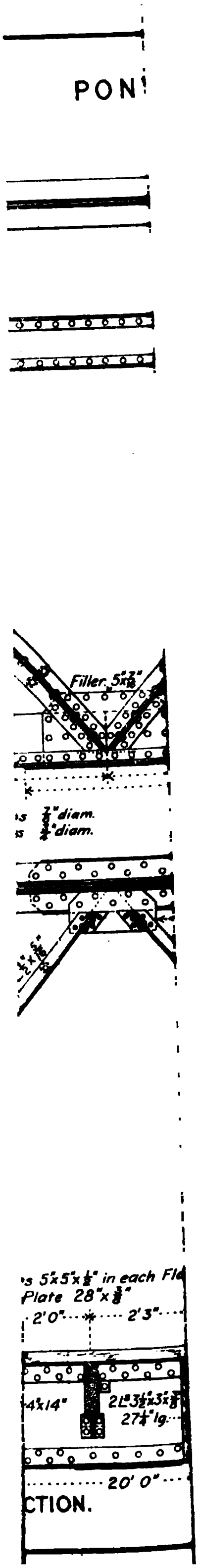
On the outside of the truss the vertical shear is 10 820 pounds, and as 7 rivets in the splice plates are available for shear the stress per rivet is $10\,820 \div (7 + 7) = 770$ pounds. The maximum stress is therefore the resultant of 770 and 3280 pounds, or 3370 pounds, which is 90 pounds in excess of the allowable pressure for a ⅞-inch field rivet. This deficiency may be made up either by lengthening the splice plate on that side and adding a few more rivets, or by riveting the splice plates to the cantilever web in the shops so that two thirds of the rivets in these plates may take a higher unit stress. The latter plan will be adopted.

The rivets uniting the angles of the sub-vertical which are on opposite sides of the web of the lower chord of the truss may now be located. Their object is to equalize the stresses in these angles and also to keep the lower flanges of the cantilever and floor beam in contact with the chord. It is seen from the drawing that at the sub-verticals four rivet holes must be deducted from the web section of the lower chord. At these places the required net section is provided by the fillers under the chord angles being made continuous between the connecting plates at *a*, *c*, *e*, etc.

If the allowable unit stress in their rivets be reduced in the same manner as that in the laterals themselves (¶¶ 59 and 60), four field rivets will be needed at each extremity of the diagonals between *a* and *b*, and three rivets for those between *b* and *d*. As the vertical legs of the angles are to be attached by means of clips (¶ 54), no less than four rivets will be used in any diagonal.

The laterals are united to the inner angles of the lower chord (¶ 50), and it is evident that the necessary size of the connecting plates will cause them to receive more rivets than the small chord increments of the lateral system require. The eccentricity of this attachment is very small and needs no special provision made for it since the live load is supposed not to be on the bridge when the maximum wind pressure occurs (¶ 48).

The angles composing the web members which are on the same side of the connecting plates will be riveted together at intervals of one foot and those on opposite sides by rivets and


PON
Filler
diam.
diam.
in each
Plate 28"
2'0"
2'3"
4"x14"
27½" lg.
20' 0"
CTION.

washers at intervals of two feet. At the ends the rivets are spaced somewhat closer. The angles will thus act together as a unit.

The general pitch of the rivets in the chords and end posts uniting their angles to the webs and cover plates is not subject to theoretic determination, and will be made uniformly $4\frac{1}{2}$ inches except at the joints, where these rivets are governed by those in the web members and connecting plates and which were previously located (¶¶ 65 and 90).

## ART. 70. SHOES AND BED PLATES.

(See Paragraphs 41–44, 70, 71, 86 and 87 of the Specifications in Art. 65.)

The reaction of the support is about 90 000 pounds and for iron rollers 3 inches in diameter requires 68.2 linear inches of roller (¶¶ 42, 86 and 87). If ordinarily good limestone be used the bed plate must have an area of $90\,000 \div 250 = 360$ square inches. For a unit stress in the outer fiber of 12 000 pounds and a uniformly distributed load of 250 pounds per square inch the bed plate may project beyond the rollers a distance equal to four times the thickness of the plate. Under the same conditions for a $\frac{3}{4}$-inch bearing plate the rollers should not be longer than about 12 inches when the transverse strength of the $\frac{7}{16}$-inch angles is taken into account. Six rollers would then be needed and would rest on a base about $17\frac{1}{2}$ inches long. The bed plate should therefore measure at least 16 by 22.5 inches, and be $\frac{3}{4}$ inch in thickness (¶ 70). To provide for the anchor bolts, etc., the plates are made 19 by 24 inches.

The longitudinal motion to be provided for is $0.0000067 \times 80 \times 12 \times 150 = 0.965$, or say 1 inch (¶ 41). Two anchor bolts $1\frac{1}{4}$ inches in diameter will be used at each shoe (¶ 70), and hence the slotted holes in the bearing plate at the roller end should be about $1\frac{3}{8}$ by $2\frac{1}{2}$ inches (¶ 43).

The tongue on the bed plate (¶ 44) may be a bar $1\frac{3}{4} \times \frac{5}{16}$ inches fastened by $\frac{5}{8}$-inch countersunk rivets, the guide bars for the rollers $2\frac{1}{2} \times \frac{5}{8}$ inches, and the bolts to keep the guide bars in position $\frac{3}{4}$ inch in diameter. As these details are of the same form as those used for the plate girder (see Plate III) they will not be shown here. It may be added that expansion rollers are preferably made of steel.

## ART. 71. ESTIMATE OF WEIGHT.

The weight of one half of the truss is now computed, and the results classified as follows:

| | Pounds. |
|---|---|
| Upper chord, with splice angle | 2 285 |
| Lower chord, with splice angle | 2 812 |
| End post | 729 |
| $2\frac{1}{2}$ sub-verticals | 886 |
| 4 diagonals (407 + 470 + 294 + 294) | 1 465 |
| Main members | 8 177 |
| Connecting plates and fillers | 643 |
| Bearing plate | 111 |
| Washers | 87 |
| Rivet heads (1059 pairs) | 470 |
| Total weight of one half of the truss | 9 488 |
| One fourth of the lateral system | 558 |
| Plates and rivets connecting cantilevers to floor beams | 210 |
| Total | 10 256 |

Deducting the partial panel load at the support due to the above weights and which equals 916 pounds, and dividing the remainder by $(\frac{3}{4} + 1 + \frac{1}{2})$, the full panel load for the truss alone is found to be 4150 pounds, which is 150 pounds more than the value assumed in computing the stresses, the excess being 1¼ per cent of the total dead panel load. This is within the limit specified in ¶ 14.

The computation of the stresses in the other truss and its design will be left as an exercise for the student, who may afterward determine whether there is sufficient difference between the sections of the main members to justify their adoption in constructing the truss or whether it would be more economical to make both trusses alike, excepting the sub-verticals with their side braces (¶¶ 38 and 39), even though it require somewhat more material.

The stresses in the lateral system were obtained by using the wind pressure specified in ¶ 18. If the specifications in ¶¶ 15 and 16 are applied to the truss and floor system designed, the pressures at the floor beams *b*, *d* and *f* are found to be 4720, 5520 and 5420 pounds respectively, the pressure on the vertical projection of the floor joists, etc., being 1020 pounds and on the vertical projection of one of the iron hand railings 640 pounds. The railing next to the street-railway track is to be attached directly to the truss and requires no posts. The center of wind pressure on the entire structure is 3.1 feet above the lower surface of the bearing plate.

The truss requires no camber (¶ 45).

# CHAPTER VIII.

## CLASS-ROOM DESIGNS.

### ART. 72. PLAN OF WORK.

THE general plan of the arrangement of designing work for a class in a technical school will depend somewhat on the number of students. For a small class the instructor will be able to give close personal attention to each student, and accordingly the exercises assigned may be quite different. For a large class, however, this plan cannot be advantageously followed, since the labor required of the instructor in checking the computations and drawings would be very great. In the latter case it may be found advisable to assign work of the same general character to all, the spans or depths being different, so that independent computations by each student are required for every part of the bridge. Thus the different computations when tabulated serve as a kind of check upon each other. It is true indeed that by this method an indifferent student may derive much assistance by inspecting the work of others, but the benefit thus derived is probably as great as if similar assistance were constantly given by the instructor in charge. A celebrated educator once said that a college student learned as much from his fellow students as from his teachers, and although this may be doubted as a general rule, it is nevertheless true that discussions among students are of the greatest value in leading them to clear perceptions of principles.

The computations and estimates of designing ought to be made directly in ink in a special book assigned for that purpose. Scribblings with a pencil on any scraps of paper that may happen to be at hand are not conducive to good work. The student should first record the main data and instructions, and then take up the various steps of the design in the designated order, reading carefully the specifications and the instructions of the text-book applicable to each step. The drawing should be begun as soon as possible, and thus the use and necessity of the computations will be better appreciated. Drawings and blue prints of similar structures will of course be at hand for consultation, and visits of inspection to actual bridges should be made. In the three following articles are given an example of a design made in the class room.

## ART. 73. DATA FOR A PRATT TRUSS BRIDGE.

The following are the general data and instructions given to each of the students in civil engineering (24 in number) in the class of '93 at Lehigh University, as one of the exercises in bridge design.

(1) The bridge shall have through intersection trusses of the Pratt type, with inclined end posts, and shall be of the pin-connected kind. All parts shall be of wrought iron or steel, except the cross ties for the track, which shall be of oak.

(2) The following are the main dimensions of the bridge: Span, center to center of end pins = 107 feet 6 inches; depth between centers of chords = 21 feet 6 inches; width in clear between trusses = 14 feet; clear head room above base of rail, not less than 18 feet 6 inches; number of panels on lower chord = 6; angle between line of track and faces of abutment = 90 degrees.

(3) For the purpose of computing the stresses in the trusses the total dead weight of the bridge in pounds per linear foot shall be found from $560 + 5.6l$, in which $l$ is the span in feet. and one third of this shall be taken on the upper

chord. The track shall be assumed to weigh 400 pounds per linear foot.

(4) The live load shall be a single consolidation locomotive and train as specified for class A on page 4 of COOPER'S Railroad Bridge Specifications.

(5) The wind pressure shall be that stated in COOPER'S Specifications, Art. 24, page 6.

(6) The working stresses per square inch for all parts of the structure shall be those given in COOPER'S Specifications, and all the details shall be arranged in accordance with the requirements therein set forth.

The span of 107 feet 6 inches above stated is the one corresponding to the design given on Plate VI. This was the longest span assigned, the shortest being 95 feet, and the difference between successive spans being usually 6 inches. In all the 24 designs the number of panels was six, and the depth of the truss between chord centers was one fifth of the span.

## ART. 74. STEPS OF THE DESIGN.

In the case of the Pratt truss under consideration the following steps or divisions were assigned to be followed in order by each student. Of course the student who uses this textbook will naturally follow the method of procedure in Chap. VI, which differs in some respects from the one here given.

(*a*) Determine lengths of panels and diagonals; also sec $\theta$ and tan $\theta$.

(*b*) Find the panel dead loads per truss.

(*c*) Compute web stresses due to dead load.

(*d*) Compute chord stresses due to dead load.

(*e*) Make a tabulation diagram for the live load (Part I, Art. 62).

(*f*) Compute web stresses due to live load.

(*g*) Compute chord stresses due to live load.

(*h*) Compute wind stresses for upper lateral system.

(*i*) Compute wind stresses for lower lateral system.

(*j*) Compute wind stresses for portal bracing.

(*k*) Make a strain sheet, showing dead, live, and wind stresses.

(*l*) Design sections of upper chords and posts.

(*m*) Design sections for lower chords, main and counter ties.

(*n*) Design sections for upper lateral struts and rods, and for lower lateral rods.

(*o*) Design of portal strut and bracket, and of panel brackets.

(*p*) Design the arrangement of joints, and of reinforcing plates.

(*q*) Find sizes of pins and eyebar heads.

(*r*) Design the plate hangers for floor beams.

(*s*) Design the floor beams.

(*t*) Design the floor stringers.

(*u*) Design connections of lateral bracing with chords or floor beams.

(*v*) Design the bed plates and rollers.

(*w*) Estimate the weight of material.

(*x*) Compare the assumed and computed weights.

(*y*) Estimate the cost of the bridge.

The above divisions of the subject were followed in order, the computations necessary in each being made in ink on the left-hand page of the record book, while the sketches and results were given on the right-hand page. In all about 50 pages of letter size were needed. After (*q*) was completed the drawing was begun, the outline panel being on a scale of $\frac{1}{2}$ inch to 1 foot, and the details on a scale of 1 inch to 1 foot. Plate VI shows the drawing made by F. C. WARMAN, which was selected as best adapted for photographic reduction.

### ART. 75. ESTIMATE OF WEIGHT AND COST.

After the completion of the drawing an estimate of the weight of the bridge was made, the form adopted being the following:

For the inclined end posts:

| | Pounds. |
|---|---|
| 4 cover plates, $\frac{5}{16}'' \times 18'' \times 28'\ 7''$. ... | 2 147 |
| 8 channels, 24 lbs., $28'\ 7''$ .......... .... | 5 491 |
| 8 cross plates, $\frac{3}{8}'' \times 15'' \times 18''$......... | 226 |
| 144 lattice bars, $\frac{3}{8}'' \times 2\frac{1}{4}'' \times 20''$......... | 677 |
| 16 reinforcing plates, $\frac{1}{4}'' \times 9'' \times 1'\ 7\frac{1}{2}''$.. | 538 |
| 16 reinforcing plates, $\frac{1}{4}'' \times 9'' \times 1'\ 7''$... | 524 |
| 8 pin plates, $\frac{3}{8}'' \times 9'' \times 1'\ 10''$. ..... ... | 166 |
| Total for 4 inclined end posts..... | 9 769 |

In this manner the weights of the several parts of the bridge were estimated, the final results being as follows:

| | | |
|---|---|---|
| Upper chords . ............. | 12 308 | pounds. |
| Lower chords................. | 8 432 | |
| Inclined end posts............. | 9 769 | |
| Vertical posts..... ........... | 4 169 | |
| Web ties ...... ............. | 9 079 | |
| Floor-beam hangers........... | 2 594 | |
| Floor beams ................. | 10 302 | |
| Floor stringers................ | 20 225 | |
| Lower lateral bracing.......... | 2 788 | |
| Upper lateral bracing . ....... | 3 455 | |
| Knee and portal bracing........ | 2 081 | |
| Pins......................... | 2 274 | |
| Rollers...................... | 636 | |
| Bed plates and standards....... | 3 566 | |
| Excess for rivet heads... ..... | 2 520 | |
| Total weight of iron........ | 94 198 | pounds. |
| Less bed plates and rollers.. | 4 202 | |
| Weight of iron above rollers. | 89 996 | pounds. |

| | |
|---|---|
| Iron dead load........... | 837.1 pounds per linear foot. |
| Specified weight of track.. | 400.0 |
| Computed dead load...... | 1237.1 pounds per linear foot. |
| Assumed dead load ....... | 1162.0 |

Computed exceeds assumed weight, 6.5 per cent.

The above method of estimating weights is an advantageous one for students, since each piece is taken up in order and the chance of omissions thus lessened. If time allows, an order sheet may also be made out, in which things of the same kind would be grouped together, but as this is merely clerical work it is not particularly instructive.

With regard to cost, class estimates are necessarily defective, and the important idea is to secure uniformity and to use prices sufficiently high to allow a fair profit. The item of erection will be especially uncertain, and the plan here adopted is to estimate it at 0.5 cent per pound. The following is the estimate made for the Pratt truss bridge of 107.5 feet span whose design has been here described and the general plan of which is given on Plate VI. At present prices the cost of iron might be taken a little lower.

| | | |
|---|---|---|
| Iron, 94 200 pounds | @ $2\frac{3}{4}$ cents............ | $2590.50 |
| Manufacture, | @ $\frac{3}{4}$ cent............ | 706.50 |
| Erection, | @ $\frac{1}{2}$ cent............ | 471.00 |

Estimated cost of bridge, 107.5 feet span, $3768.00

and hence the estimated cost per linear foot, exclusive of track, is $35.05.

The detail drawings for a bridge of this kind, as prepared in the office of a bridge company, would embrace four or five sheets in addition to the general plan shown on Plate VI. The time devoted to the subject in technical schools is not sufficient for these to be drawn, and indeed it is scarcely necessary that they should be undertaken by a student in a course whose object is to teach principles rather than a special trade. It may safely be affirmed that a student who carefully makes the computations and general plans of three or four different kinds of bridge structures has acquired a training in the application of principles which will fit him to enter a bridge office and do good work. Yet it should be said here to the student that what he has learned in the school is but a trifle when compared to that which he will have to learn in order to become a competent bridge engineer.

## ART. 76. COMPARISON OF CLASS DESIGNS.

When the above plan of class work is followed all the computations of stresses, and many others, may be checked by tabulating the results and comparing the differences for successive spans. Owing to differences of judgment and to inexperience, however, the final weights will not vary according to definite laws as would perhaps be the case if the same designs were made by an experienced man. Yet upon comparison of such results it is often found that the percentages of variation are within fair limits. For example, in the twenty-four designs of the Pratt truss described in the preceding Articles the mean excess of computed over assumed weight was about 5 per cent, seventeen being less and seven greater than this, and the greatest excess being 14 per cent. Four men made the computed weight less than the assumed, the lightest being 5 per cent lower. Neither the computed weights nor the per cents of excess show any signs of systematic variation with the length of span when considered in order one by one, but when arranged in groups traces of such are discerned in the means. For example, five of the shorter spans compared with five of the longer ones give the following results:

| Span in Feet. | Pounds per Linear Foot. Assumed. | Computed. | Percent of Excess. |
|---|---|---|---|
| 95 | 1 092.0 | 1 178.6 | 8.1 |
| 95.5 | 1 094.8 | 1 040.0 | −5.0 |
| 96 | 1 097.6 | 1 204.0 | 9.7 |
| 96.5 | 1 100.4 | 1 155.0 | 5.0 |
| 97 | 1 103.2 | 1 193.5 | 8.2 |
| Means | 1097.6 | 1 154.2 | 5.2 |
| 104 | 1 142.4 | 1 202.5 | 5.2 |
| 104.5 | 1 145.2 | 1 305.3 | 14.0 |
| 105 | 1 148.0 | 1 218.0 | 6.1 |
| 105.5 | 1 150.8 | 1 110.5 | −3.5 |
| 106 | 1 153.6 | 1 197.4 | 3.8 |
| Means | 1 148.0 | 1 206.7 | 5.1 |

Here the mean computed weights of the two groups show a difference of 52.5, while that of the assumed weights is 50.4, the percent of increase being closely equal for the two. This agreement is a closer one than can generally be expected in designs made under such circumstances.

In conclusion it should be noted that the training of a student will be more effective the more he learns to think for himself. Constant application to the instructor in charge to settle doubtful questions may be the quickest way to advance the work of design, but it is rarely the best way for the good of the student himself. The judicious instructor will usually answer such questions by asking others which lead the student to see the principles involved and thus enable him to make his own solution. It is indeed often better for a student if he happens to follow a wrong method which requires two or three pages of his computation book to be rewritten than if he were kept constantly on the right track by the admonitions of the instructor. The student who makes and discovers mistakes learns to avoid them, while he who works correctly under constant supervision is liable to fall into error when thrown upon his own resources.

The number of designs to be made by each student in the class room should not be less than two. The class whose work has been mentioned above made three designs, a plate-girder bridge, a riveted lattice bridge or a roof truss, and a pin bridge, the total time employed being about 85 exercises of two hours each. Some students also made more extended designs for theses.

## ART. 77. EXERCISES AND PROBLEMS.

The following exercises and problems, which are arranged with particular reference to the subject matter of this volume, indicate how a great variety of problems may be devised for class use. But others far more advantageous to students may be stated in connection with actual bridge structures which are convenient for them to measure and sketch. The numbers prefixed to the exercises refer to the corresponding Chapters.

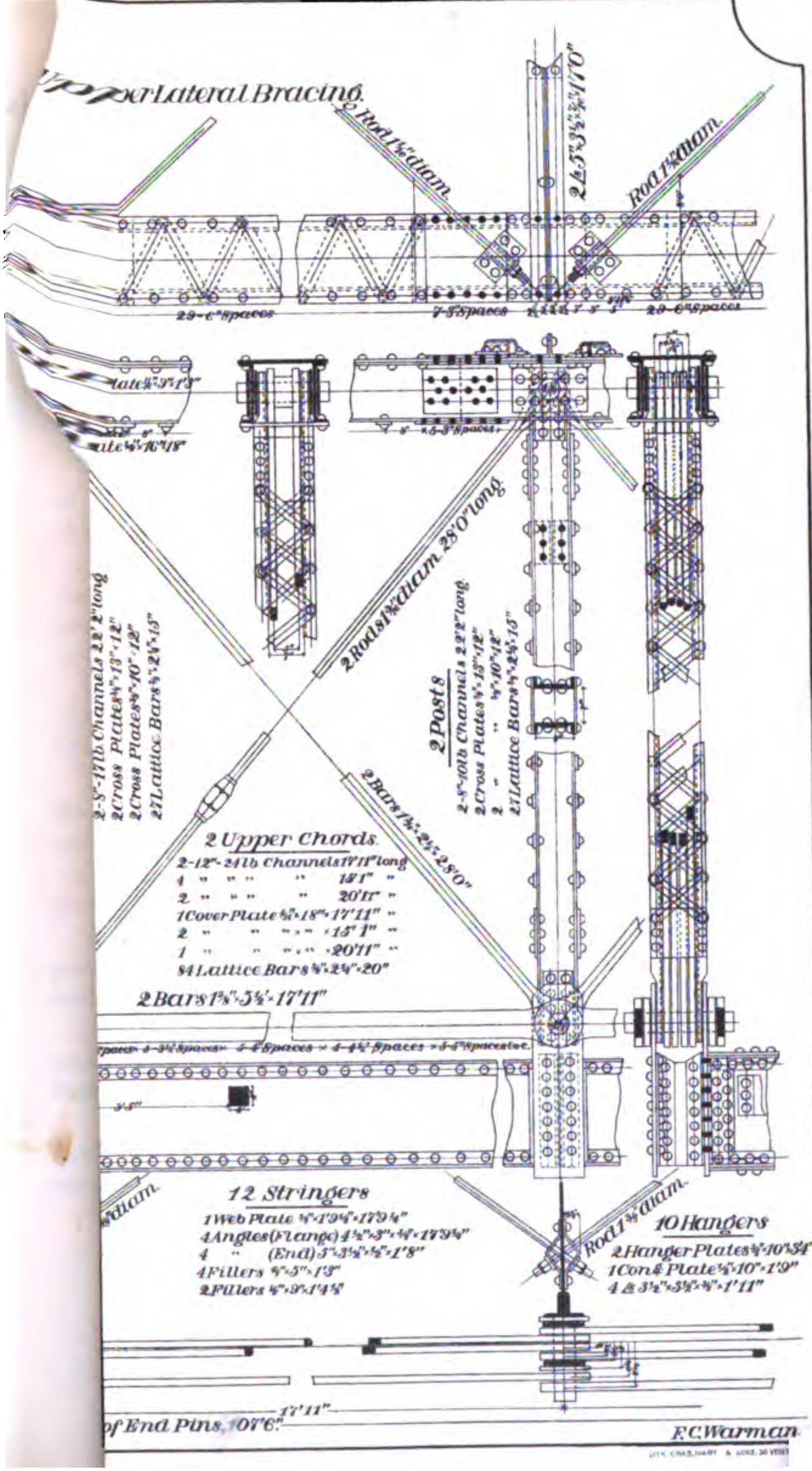
I. Make a list of the articles on the history and design of bridges which have appeared in Transactions of the American Society of Civil Engineers since 1880, with a synopsis of the contents of each article.

II. Make a theoretic comparison of a single-system Warren truss with a Baltimore truss, each having ten panels, and determine the theoretic economic depth in each case.

III. Make out bills of material for two railroad-bridge floors in your neighborhood, compute their weights, and compare them with the Pennsylvania Railroad standard given in Art. 19.

IV. Design a roof truss of 65 feet span and $16\frac{1}{4}$ feet rise, using the specifications and methods of this Chapter.

V. Revise the design of the plate girder in Chapter V, using a $\frac{9}{16}$-inch web, with web sheets 5 feet longer than before, the specifications being modified so as to allow the web also to take moment.


Upper Lateral Bracing.
Rod 1⅛" diam.
2 L5"×3½"×⅜"×17'0"
Rod 1⅛" diam.
29-6" spaces
7-3" spaces
29-6" spaces
2-8"-11lb. Channels 22'2" long
2 Cross Plates ⅜"×13"×12"
2 Cross Plates ⅜"×10"×12"
27 Lattice Bars ⅜"×2¼"×15"
2 Rods 1⅛" diam. 28'0" long
2 Posts
2-8"-10lb. Channels 22'2" long
2 Cross Plates ⅜"×13"×12"
2 " " ⅜"×10"×12"
27 Lattice Bars ⅜"×2¼"×15"
2 Bars 1½"×2½"×28'0"
2 Upper Chords
2-12"-24lb. Channels 17'11" long
4 " " " " 18'1" "
2 " " " " 20'11" "
1 Cover Plate ⅜"×18"×17'11" "
2 " " " ×15'1" "
1 " " " ×20'11" "
84 Lattice Bars ⅜"×2¼"×20"
2 Bars 1⅝"×5½"×17'11"
12 Stringers
1 Web Plate ⅜"×1'9¾"×17'9¾"
4 Angles (Flange) 4½"×3"×⅜"×17'9¾"
4 " (End) 3"×3½"×⅜"×1'8"
4 Fillers ⅜"×5"×1'3"
2 Fillers ⅜"×9"×1'4½"
Rod 1⅛" diam.
10 Hangers
2 Hanger Plates ⅜"×10"×3'4"
1 Con. Plate ⅜"×10"×1'9"
4 L 3½"×3½"×⅜"×1'11"
17'11"
of End Pins, 107'6".
F. C. Warman.

VI. Design a pin bridge of the same type and character of details as that in Chapter VI, the depth being changed to 28.5 feet and the number of panels increased to seven.

VII. Design a riveted lattice bridge of 90 feet span, using the specifications of the Norfolk and Western Railroad given in Chapter XIII.

VIII. Design four bridges of the Pratt type having spans of 108, 112, 116 and 120 feet, following in each case the instructions and specifications of this Chapter.

IX. Visit the works of a bridge company and write a report of about 1000 words, with sketches, describing the different shops and buildings.

X. Compile from technical papers a list of bridge lettings during the year 1890 and another for 1895, giving for each case the kind of structure, names of bidders, amounts of bids, and the final cost per linear foot for the superstructure.

XI. Visit the works of a bridge company and write a report of about 1000 words describing the machinery and the work in progress at the time of the visit.

XII. Design a ballast-floor bridge of 60 feet span under the specifications and methods of this Chapter, but using trapezoidal troughs instead of rectangular ones.

XIII. Design two skew bridges of the style and dimensions given in Art. 107, one having riveted and the other pin trusses.

XIV. Design, under the specifications of this Chapter, two pony truss bridges for a single track, the spans being 96 feet and 108 feet.

XV. Design two through Pratt bridges, the dimensions and specifications being the same as in this Chapter, except that the heights of trusses are 24 and 28 instead of 26 feet.

XVI. Design a through Baltimore truss bridge, using the specifications and dimensions of this Chapter, but taking the height of the truss as 32 feet 2¾ inches.

# CHAPTER IX.

## BRIDGE LETTINGS, AND OFFICE WORK.*

### ART. 78. SPECIFICATIONS.

WHEN a bridge is to be erected the engineer representing the party having the matter in charge should prepare rules regarding the loads to be used in the computations, the permissible unit stresses, the quality of the materials, and the details of construction. These rules are called specifications, or often "the specification." All the plans submitted are to be in accordance with these specifications, which are afterwards made a part of the contract between the buyer and the contractor.

Specifications cannot be successfully prepared except by an engineer of experience. Many railroad companies have their standard specifications, while others adopt those of COOPER. In highway-bridge work it sometimes happens that county commissioners or town authorities advertise for proposals without having definite specifications, but the result is sure to be that a poor bridge will be erected. Any one can buy a bridge, but only an engineer can do so and obtain both a stable and an economical structure. The highway-bridge specifications of COOPER and those of WADDELL are excellent guides to follow, and they can easily be obtained in pamphlet form. Parts of COOPER'S railroad-bridge specifications are given in Chap. VI, and parts of WADDELL'S highway-bridge

* A large part of this chapter was prepared by RALPH M. WILCOX, Instructor in Civil Engineering in Lehigh University.

specifications in Chap. VII, while in later chapters will be found in full those of several railroad companies.

The following extract from a lecture by THEODORE COOPER at the Rensselaer Polytechnic Institute in 1893 states in an excellent manner the fundamental purpose of specifications:

"Engineering specifications are the detailed particulars supplementing and explaining the plans of any piece of work, and governing the character of the materials and the methods of construction to be used for the same; or where unaccompanied by plans, are the rules and directions under which any work is to be designed and executed.

"Their purpose is a twofold one. First: They are to enable bidders upon any work to understand fully the character and extent of the work and what they are expected to furnish and what to do; in order that they may be able to make suitable estimates upon which to formulate an intelligent and proper bid. Second: They are, in connection with the plans, to serve as the reference in regard to all questions as to qualities of the materials and workmanship during the execution of the work, in order to avoid misunderstandings between the engineers and contractors; the contractor not being allowed to furnish poorer or less suitable materials and workmanship than is there specified, nor the engineer to demand any better without giving an extra compensation. Nothing serves better to obtain the best class of contractors and to obviate much of the friction which occurs during construction between the engineer and the contractor than a good specification, carefully and clearly expressed. A loosely-drawn and incomplete specification is always attractive to the worst class of contractors, or those who do not intend to do an honest job and who will take advantage of every weak point to get all they can out of the work."

The following advice from the same lecture should be carefully heeded by young engineers who are called upon to prepare specifications:

"As the preparation of an original specification requires a large fund of practical experience and observation, it is better that your early efforts in this line be based upon well-known existing specifications, modified to suit your special needs. Before doing this, carefully study the specification, which you may select as your model, that you may have a clear understanding of the intent and meaning of its several clauses, and of the theory upon which it is based.

"Be careful to consider the specification as a whole, and not solely as a collection of disjointed clauses, part of which you may select at random, and part reject. Frequently the various clauses are so correlated that taken separately the intention of the specification is lost. By comparison of different specifications of well-recognized merit, you will frequently find that while each may produce equally good results, they accomplish this purpose by different methods or on different theories. By endeavoring to compile a common specification from these several specifications without proper judgment, as is not unfrequently done, a worthless and often absurd result is produced.

"In compiling a specification by the use of old and clearly-defined clauses, do not, without good reason, attempt to change their wording. You may reduce the force of such clauses by permitting a new interpretation, and lose the strength of an accepted interpretation by other engineers and contractors.

"A specification is not intended to be a treatise on the refinements of engineering theory, but a practical set of rules and conditions upon which work is to be let and constructed. A specification should therefore be reasonably brief and positive."

### ART. 79. ESTIMATES AND PROPOSALS.

After the preparation of the specifications, proposals or bids are invited from bridge companies for the manufacture and erection of the structure. In general bridge companies contract for and build only the superstructure, while the piers and abutments are erected by masonry contractors.

The usual mode of procedure is to publish an advertisement which gives the location, length, number of spans and width of the bridge, stating whether highway or railway, and whether timber, stone or metal is to be employed. The advertisement mentions where specifications can be seen and information obtained, and names the day and hour when the proposals will be opened. It often states that a certified check for a certain amount must be deposited by each bidder as a guarantee that he will enter into a contract in case the work is awarded to him. Bidders are invited to be present at the opening of the proposals, and the right is reserved to reject any or all bids. It should also be required that each bidder shall present a strain sheet and a general plan of the structure that he purposes to erect.

A bridge company which desires to put in a bid for building the bridge sends one of its agents to the place to procure all the data available. Sometimes the engineer in charge of the work has plans prepared on which the companies estimate and bid, but usually each company prefers to make and submit its own plans. The agent examines closely the locality and estimates the cost of hauling the material from the nearest railroad station, as also the cost of erection. The latter item is often an uncertain one, since delays due to the weather or to floods in streams are liable to arise, and sometimes accidents occur which cause the loss of all profits. It should also be the duty of the agent to become acquainted with the parties who purpose to build the bridge, so that in case of a close competition he may be better prepared to induce them to accept the proposal of the company which he represents.

The computations and designs made by a bidder in order to estimate the cost of a structure are similar to those given in the preceding and following chapters. The style and proportions of the bridge being decided upon the stresses are computed by the methods of Part I or Part II, and a strain sheet is prepared showing these stresses and the sections of the main members. A general drawing is also made showing elevation, plan and cross section, with the main features of all details. From this drawing a bill of material is made out, and estimates of the weight and cost of manufacture are prepared. Adding to this the estimated cost of freight and erection, and a fair percentage for interest on invested capital, profit and contingencies, the bidder decides upon a sum to state in his proposal.

The usual practice in highway-bridge lettings is for each bidder to offer a lump sum for the erection of the superstructure ready for traffic and painted. On railroads it is often the case that the cross ties, rails and guard timbers are laid by the railroad company, so that the lump sum is exclusive of the track. On some railroads, however, the proposals are required to be made per pound of the finished structure ready for the track, and in such cases the actual sections of the members are not allowed to exceed by more than 2 or 2½ per cent the theoretic sections as required by the stresses and specifications.

### ART. 80. LETTINGS AND CONTRACTS.

At the hour stated in the advertisement the proposals are opened and read in the presence of the bidders. The accompanying plans are referred to the engineer in charge to see if they conform to the specifications. It is, however, usually only necessary for him to check the computations of two or three of the lowest bidders if their plans seem otherwise acceptable. On the receipt of the report of the engineer the commissioners or authorities in charge make a formal award of the work to the lowest responsible bidder whose plans are satisfactory, and he is notified to appear and sign the contract, while the plans and certified checks of the other bidders are returned to them.

It often happens at a bridge letting that the highest bid is about double the lowest. This wide discrepancy is probably due more to the fact that certain companies have better facilities regarding freight and erection than to the relative economy of the several types of trusses. The number of proposals submitted for a structure usually ranges from five to twenty.

This method of bridge lettings, in which each bidder offers his own designs, has many advantages, but it has the disadvantage that only one out of a number of plans is utilized. If twelve bidders each spend $100 in making estimates and designs for a single bridge there has been expended altogether $1200 which in some way must be paid by the buyers of bridges. It is not an infrequent practice, indeed, that the twelve bidders form a pool, each adding $1200 to his bid, and then the successful bidder pays $100 to each of the eleven unsuccessful ones. This is a necessary evil of the method, perhaps, but the evil is not as great as often assumed, since the expenses of the bridge companies must be paid in some other way if not in this. The expenses of estimating would be lessened if the bidders were limited to plans and designs made by the engineer in charge, but in such cases it usually happens, owing to details of construction, that their bids are higher than for their own designs. Open competition has been one of the elements which have led to the present economic forms of bridge trusses (Chap. II), and, notwithstanding the necessary evils of pools, its results continue in general to be satisfactory.

The contract which is entered into between the parties and signed by both specifies that the bridge company shall erect the structure according to the plans and specifications and that the other party shall pay to said company a certain amount for the same. It also sets forth in detail the conditions regarding time of completion and payment, the liabilities of the contractor for damages due to accidents, penalties for delay of completion, and other conditions mutually agreed upon. When this document is signed both parties are legally bound by its provisions and the bridge company is ready to begin the detail drawings for the shop work.

A bond is also required to be given by the contractors, signed by them and two responsible bondsmen, binding the contractors under a penalty to execute the contract in pursuance of its terms and conditions, and in accordance with the plans and specifications thereunto annexed. This bond is in law of the nature of a promissory note, and in case of default of the contractors an action at law can be brought to recover the sum stated therein, or such part of it as may be sufficient indemnification for the damages sustained.

There are many engineers who own no bridge works, yet nevertheless bid for and take contracts to erect structures. Such men have arrangements with bridge builders to manufacture their bridges at certain prices per pound, or they make special bargains for the contracts that they secure. Many of these engineers do good work and make a fair profit. Chapter XIX illustrates a bridge erected under this method.

### ART. 81. DRAFTING-OFFICE PRACTICE.

After the contract has been secured, working drawings have to be prepared for the benefit of the templet makers, shopmen, and the workmen in general who have anything to do with the construction of the bridge. These drawings are made in the drafting room, which is, or should be, well heated, well lighted, and well ventilated. In the drafting rooms of some of our larger bridge-building companies there are as many as twenty-five or thirty men, all under the immediate charge of a superintendent or head draftsman, who is thoroughly posted on all kinds of detail work and shop methods. Each man is supposed to be supplied with a complete outfit of drawing tools, and to have a desk to himself, with drawers for paper, tracing linen, and tools. The old style of drawing desk is flat on top and from three feet six inches to four feet in height, with a regular drawing board on top. The more modern desks are not quite so high, and the top is so arranged that it can be tipped up toward the draftsman, making it easier to see and get at all parts of the drawing, and at the same time doing away with the fault of lying down on the table in order to get at the parts near the top of the drawing. The drawing board in this latter case is the top of the desk.

A large supply of drawing materials is necessarily kept on hand by the company. Adjoining the drafting room is a fire-proof vault in which are kept all plans and drawings of structures that have been built by the company. These are of great value to the company and also to every draftsman. The vault is the draftsman's library. In consulting it he may find many unique and useful designs, especially in detail connections, which will help him out of difficulties.

All drawings, before they leave the drafting room, pass through the hands of a man who looks over them very carefully and checks the work in every particular. If any mistakes are found they are duly marked, in which case they are returned to the draftsman and he makes the necessary corrections. As the shopmen have to adhere strictly to the drawings in their work, the preparation of working drawings necessarily requires great accuracy and clearness, in order that there shall be no delay or failure in the parts of the bridge being put together.

As a general thing in bridge works the draftsmen who make the working drawings have nothing to do with the computation of stresses in the members. That work is done by the computers, who make the designs of structures and the estimates of cost. These are men of experience and consequently less liable to err.

The data which the draftsman receives from the computers consist then of the skeleton diagram of the structure to be drawn, the stresses in the members, and perhaps the sections to be used, and a copy of the specifications.

The first thing the draftsman does is to find out what material is required and how much of it. If the structure is of considerable size, this is best done by laying out the work in a general way on thick brown paper prepared for this purpose, not stopping to put in the details, but going far enough to enable him to determine quite closely what are the lengths and sizes of the angles and plates which are required.

After having ascertained what material is required the draftsman consults the list of material in stock, and if he finds any in his bill that is not in stock he makes out an order list, from which the material is ordered immediately, for it must be on hand as soon as the drawings are finished.

The drawings already laid out in a general way are now completed by placing tracing linen over them and tracing the work from the paper and filling out the details on the tracing linen. The size of the completed drawing is about twenty-six by forty inches, with a single heavy border line drawn around the sheet one half inch from the edge. The scale used is about one and one quarter or one and one half inches to the foot. If there is very little detail work to be shown, one inch to the foot is often used.

The work is so arranged on the drawing as to leave a space in the lower right-hand corner for the title. This should give a full description of the drawing, including the number of the contract, the location of the bridge, the number of the sheet, the name of the draftsman, and the date.

The tracing is done on the back or unglazed side of the linen. This side shows pencil lines much better than the glazed side, and it will take ink lines just as well.

If it is necessary to do any erasing on the tracing linen a rubber ink eraser is used carefully and patiently. The erased area is then rubbed with a stick of pumice stone before inking again, to prevent the ink from spreading. The point of a knife or other sharp tool should not be used to erase lines or spots from tracing linen which have to be inked over again. If the surface of the tracing linen becomes greasy so that the ink will not take well, a little powdered chalk, sprinkled on and rubbed carefully with a cloth, absorbs the grease and gives a better working surface.

No fixed rules can be given as to just how the drawings for a bridge shall be arranged, as different companies have different rules, and no two men will arrange their work alike or make the drawings for the same bridge in the same way; but the draftsman should bear in mind constantly that the object of a working drawing is to show the workmen how to make the parts of the bridge and how to put them together. With this in mind he should have some system about the arrangement of the work on the sheets; for example, compression members such as chords or posts should be drawn in one group, eyebars, ties and counter ties in another, floor beams and stringers in another, and so on.

The drawing of each piece should be made so plain and complete that the workman may clearly and easily understand it. The dimensions of all pieces, rivet spacing and pitch of rivets should be given in full on the drawings. All printing should be plain and well done, though time should not be wasted in this work. All figures should be large enough to take and show well in the blue print. If the space on the drawings between the rivet heads will not permit of good-sized figures being placed in them, then lines should be projected off to one side of the member and the figures placed between them. Arrow points should be placed at the points between which the distance is given. Quite heavy lines should be used so as to give a good clear blue print. Fine lines should be avoided except for dimensions.

After the drawings have been completed in every detail a bill of material is prepared. This is made in such a way as to serve as a shipping list also. It contains a list of every individual piece entering into the structure. These are arranged in groups in the list just as the pieces are assembled to make up a member.

All pieces which require forging, such as eyebars, ties and counters, are listed also on a sheet known as the forge sheet. Full dimensions and details, and perhaps sketches, are required on this sheet, which then goes to the forge shop. A list of all field rivets and bolts is also made which gives their size, length, grip, and their location in the bridge (see Art. 121).

After this listing has been done both drawings and lists go back to the computing room, where every item, line and figure is carefully checked. If no errors are found, which is rarely the case, the draftsman may consider his work completed. The number of sheets of drawings required for a span of moderate length varies from five to fifteen. In Art. 103 will be found a list of the sheets made for a lattice bridge of 81 feet span. Blue prints are next taken from the tracings, and the work is ready for the shop (Chap. XI).

# CHAPTER X.

## BRIDGE SHOPS AND BUILDINGS.*

### ART. 82. GENERAL CONSIDERATIONS.

A SHOP is a room or building devoted to work. It generally contains tools and machinery which are run by power, either steam, compressed air or electricity. All the buildings of a bridge-building establishment, together with the tracks, cranes and other appurtenances, constitute the plant. This plant may include many shops connected by tracks, or the whole plant may be under one cover, according to the kind and amount of business done by the company. Usually each building is a distinct shop in itself, with its foreman in charge, the whole plant being under the control of a general superintendent.

This chapter treats of the practical design and arrangement of buildings for carrying on most advantageously the construction of wrought-iron and steel bridges and other structural work. In the design and construction of manufacturing plants the three great objects to be kept in view are: 1st. The minimum cost of manufacture of product; 2d. The minimum cost of maintenance and repairs; 3d. The lowest first cost of building.

Many manufacturing corporations in starting their plant reverse the order, and reduce the first cost of building to the absolute minimum, neglecting the economy in manufacturing, and in the maintenance of their plant. This method of procedure is sometimes desirable in the manufacture of a class

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* Written by CHARLES M. JARVIS, M. Am. Soc. C. E., President of the Berlin Iron Bridge Company.

of goods where the profits are large and the working capital at the outset of the enterprise limited, as in time they can replace their cheap wooden buildings and uneconomical methods of manufacture by more modern appliances; the expense of these changes to be borne by the profits in the business. Even in this case if account is taken of the reduced cost of maintenance together with the reduced cost of production of the manufactured product, the saving would pay a large interest on the original cost of the best class of construction.

The present tendency of bridge companies is decidedly in the direction of large, well-lighted and well-heated, roomy buildings, constructed of iron or steel. Employees appreciate a shop of this character and will turn out enough more product to pay the increased cost of the investment.

The old dingy machine shop with its low wooden roof, its small dirty windows, its impure air through lack of ventilation, is being gradually abandoned for the handsome modern construction which insures both light and ventilation to the workmen, and at the same time reduces the insurance risk and the cost of maintenance to the absolute minimum.

The plant of a bridge company is divided as follows: the Forge Shop, the Machine Shop, the Pattern or Templet Shop, the Truss Shop, and often a Foundry. Another shop is sometimes added, known either as the Railing or Shutter Shop. In this the machinery is light and adapted to small work, such as bridge railings for sidewalks, corrugated-iron doors, shutters and fire escapes.

## ART. 83. THE FORGE SHOP.

The Forge Shop is generally divided into three departments: the Rivet-making Shop, the Shop for Small Forgings, and the Eyebar Plant.

In the Rivet-making Shop are placed the furnaces for heating bridge rivets and the machinery for heading them. This machinery consists of one, two or three ordinary heading machines, a sorting board (for assorting the rivets and bolts) and a scale for weighing the finished product. The bar iron from which the rivets are made is stored near by, and the finished product is carried out on a car having a box top, so arranged as to receive boxes holding a given weight of rivets.

The tools in the Shop for Small Forgings consist of a small steam hammer, perhaps a drop hammer, and sometimes a power spring hammer. With the small steam hammer the ordinary class of forgings are made. This can be done with great advantage if the hammer is equipped with proper dies and provided with a small crane or overhead traveler for handling the work. Care should be taken in designing these latter, in order to make them heavy enough to withstand the strain from a foul blow from the hammer as well as the weight of the forging to be carried. The small spring hammer is used for forging punches, drift pins, small bolts,—in fact the small spring hammer is used almost entirely for the manufacture of the tools required in carrying on the business,—its uses are innumerable, and only limited by the number of dies.

In this portion of the plant is located the upsetting machine for enlarging the ends of round and square bars. A perfect plant is equipped with two upsetting machines—one for small work, up to a size of 2½″ round and square iron, and a larger upsetter capable of upsetting the largest sizes of eyebars.

The Eyebar Plant usually consists of an upsetter for enlarging the heads, the steam hammer for shaping the heads in properly prepared dies, and the necessary furnaces for heating the material. Where steel eyebars are made an annealing furnace is also required.

In the construction of a forge shop great care must be taken in ventilating the building, for here more than in any other part of the works are foul gases, smoke, cinders, and all the refuse products of combustion combined in the greatest degree. For smoke and gas ventilation may be secured through the ridge of the building, but in order that the smoke, gases and foul air may pass out of a ventilator on the ridge of the building it is necessary that fresh air have some means of entering the building near the floor line. A forge shop should never be designed with a flat roof,—the greater the pitch of the roof the more easily can the interior of the building be ventilated, and ventilation is one of the most important elements to be secured in a forge shop. The best designs for this class of buildings have the sides arranged with large windows which are set in frames and hung with weights, so as to be easily raised and lowered, thus admitting fresh air from all sides. In the summer months the heat is intense, but by the adoption of proper appliances the comfort of the workmen may be very materially increased, as well as the amount of product which they are able to turn out in a given length of time.

## ART. 84. THE MACHINE SHOP.

The machine shop of a bridge building plant is used for the construction of the parts of bridges requiring machine work, and for the building and repair of the heavy machinery to carry on the business. Every bridge-building plant should be equipped with a machine shop capable of making its own repairs, and as the machinery about a plant of this kind is large and heavy, the appliances of the machine shop must be adapted to this work. The machinery consists of lathes, from 12- to 60-inch planers and shapers, and also a large drill press—what is known as a radial drill being preferred, the head so arranged that it moves in and out on a radial arm, thus adapting it for nearly all kinds of work. A large number of smaller lathes with strong frames are required, as the pins which connect all pin-connected bridges at the panel points are turned in this department. Here also the expansion rolls are turned. In fact all the finer and smaller parts of the bridge are finished in the machine shop. If the company operating the plant is largely employed in the construction of iron buildings, their machine shop should be equipped for building traveling cranes of the smaller sizes, as well as all jib cranes. It is almost impossible to anticipate the demands which will be made upon a bridge machine shop, as the repairs are often of a nature to severely tax the ingenuity of even the best mechanic and the appliances of the best equipped shops. All the larger tools should be provided with overhead travelers arranged to move heavy castings to and around the machines. The building should be thoroughly lighted in all its parts, and the proper way to secure this light is by means of skylights in the roof. Great care must be taken in the construction of these skylights to prevent leaking around the sides and top, but there is no method of lighting the interior of a building so thoroughly and uniformly with well-diffused light as with skylights placed in the plane of the roof. Ventilation in a machine shop is secondary to light, and can be easily secured in the ordinary manner.

## ART. 85. THE PATTERN AND TEMPLET SHOP.

In the Pattern and Templet Shop the bridge is first laid out in wood. All the machinery is designed for wood working, and consists of buzz saws, both for ripping and for crosscutting lumber, planers for smoothing the faces and edges, and the ordinary jack-borers for drilling the holes in the wooden templets. The floor of the templet shop should be made of the best soft pine, well laid, with close joints, the surface being planed smooth and even after it is laid. It is on this floor that the greater part of the work is laid out. In some shops it is customary to locate on the working drawings all rivet spacing, all holes, all angles and bevels. In other shops it is customary to show these details in a general way (the centers being carefully indicated) and the work is laid out in wood, full size, exactly as it will appear in iron. The latter method has the advantage that the work is seen in full size, and errors in connections which might escape the drafting room very often appear here. But this requires a large building and very extensive floor room. By the former method, only bench room is required. The templets should be made of soft pine, generally not less than $\frac{7}{8}$ inch in thickness, and it is of the highest importance that the lumber should be well seasoned; or otherwise templets will shrink before they are used and thus throw the connections out of line. It is the practice of most bridge companies to carry templet stock for one or two years in advance of using, in order to insure its being well seasoned.

The Railing and Shutter Shop is equipped with small punches and drill presses, and is designed only for light work, such as railings for bridge sidewalks, corrugated-iron doors and shutters, small gates, fences and fire escapes. The machinery is light, the space required for operating is small, and the cost of maintenance of this portion of the plant is less than any other. The building should be so arranged as to be easily accessible, for although the separate parts of the product are small and easily lifted, in the aggregate they amount to considerable. It should be connected with the other portions of the plant by tracks and cars for moving materials in quantity to and fro.

## ART. 86. THE TRUSS SHOP.

The truss shop, or main building, of a bridge-building plant is the largest and most important of all the buildings. It is here that all the heavier parts of the work are manufactured, and usually this portion of the plant comprises at least two thirds of the entire floor space. Some bridge-building corporations manufacture only riveted work, and in a plant of this kind the truss shop is practically the entire plant. The main or truss shop of The Berlin Iron Bridge Co., at East Berlin, Conn., is one of the best in this country, as no pains or expense has been spared to make it a model for all classes of truss work. A description of this will therefore give the student a good idea of a similar class of buildings, as it is provided with all modern appliances,—the raw material entering at one end, and going out at the opposite end as finished product.

Fig. 46 shows a general plan of the layout of this portion of the plant. The main building is 80 feet in width and 400 feet in length. On each side, extending the full length of the yard, is a standard-gauge track for delivering raw material. The shop stands in nearly a north and south direction. Shape iron is delivered by the standard-gauge track on the west side. Plate iron is delivered by the standard-gauge track on the east side. Both tracks are laid on the surface of the ground, so that raw material is easily discharged from the cars upon the adjacent skidways. Besides the standard-gauge tracks on each side of the shop, there are also standard-gauge tracks at the finishing or south end extending into the building about 120 feet, thus facilitating loading in stormy weather. When pieces of extraordinary weight are to be shipped, they can be much more easily handled by means of travelers connected with the roof trusses than by the yard derricks. The track arrangement is nearly perfect. Ten cars of raw material can be easily unloaded and stored in ten hours, and the same number of cars of finished product can be loaded and switched out at the finishing end in the same length of time.

Adjacent to the standard-gauge tracks and parallel with them are other tracks of narrower gauge. These are placed in such a position that the raw material is easily accessible and can be loaded on small truck cars with the least possible expenditure of time and labor. The points of comparison may be understood by reference to the general plan in Fig. 46. The student will

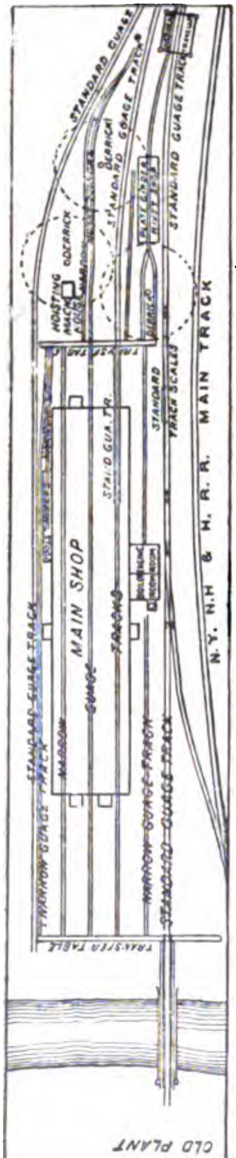
MAIN SHOP
N. Y. N. H & H. R. R. MAIN TRACK
STANDARD GUAGE TRACK
NARROW GUAGE TRACK
TRANSFER TABLE
OLD PLANT

FIG. 46.

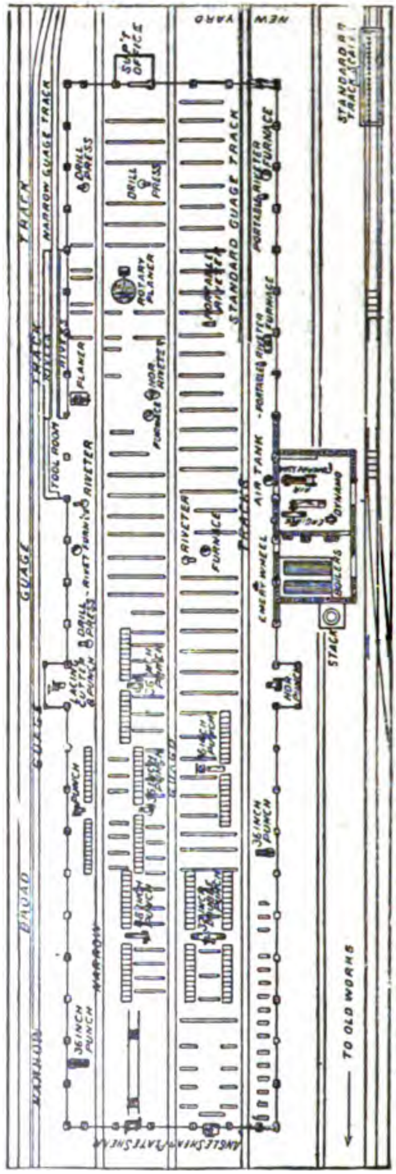
NARROW GUAGE TRACK
STANDARD GUAGE TRACK
SUP'T OFFICE
TOOL ROOM
DRILL PRESS
ROTARY PLANER
AIR TANK
EMERY WHEEL
STACK
NEW YARD
TO OLD WORKS

FIG. 47.

remember that the Mattabessett River therein shown is at the north end of the plant. A transfer table connects all the narrow-gauge tracks, so that cars can be transferred from one to the other. The main shop is equipped with three narrow-gauge tracks, one on each side (about 15 feet from the outside of the building), and one directly through the center of the building. The raw material is first loaded upon narrow-gauge truck cars and carried into the laying-out shop, where it is moved *en masse*, by means of overhead travelers attached to each roof truss, to the laying-out benches. Here it must be handled in detail, as the exact size and position of every bolt and rivet is to be carefully stamped upon it. From the laying-out benches material goes at once to the shear, where it is handled by means of jib cranes.

Fig. 47 is a plan of the truss shop, and shows the general arrangement of the machinery. The boiler and engine room is on the west side in a brick building connected with the main shop, but separated by brick walls. The fuel used is crude oil, the tanks being located on the west side of the main track of the New York, New Haven & Hartford Railroad, and placed for safety 10 feet underground. The engine room is adjacent to the boiler room, and contains not only the main engine which furnishes power for all the machinery, but also an air compressor, a blast machine, two electric-light dynamos, and a large engine for blowing hot air into the main building for heating purposes. All heavy material enters the main shop through the middle track. The cranes for the plate shear and also for the angle-iron shear are arranged to take the material from the center track, shear it or clip the ends, and deliver it to a truck car on the outside tracks. These cranes are designed to lift the material *en masse* or in detail—it being handled both ways. In this way the plates start on their course through the shop on one side and the shape iron on the opposite side. The punches are so arranged that the material after being punched and trimmed is delivered on skids adjacent to the center track, ready for setting up and fitting. Over the punches the roof trusses are designed for a capacity of three tons, and are arranged with a trolley on each truss for transferring material across the shop. The larger punches are also provided with a small traveling crane of three tons capacity for handling long heavy material. Nearly all the material is handled at the punch presses by hand, as it is found much quicker, the material being in small pieces. After leaving the punches the material is assembled ready for the riveters, and here it is necessary to provide power appliances for handling, as it now begins to assume the large proportions which it will have in the finished structure.

The riveting appliances consist of two vertical riveters, one horizontal riveter and three portable riveters. Where work weighs two tons or less it is generally handled at the stationary riveters, proper appliances being provided for moving the work cheaply and expeditiously. The appliances consist of overhead travelers, with a motion both ways; that is, a motion across the shop on the roof trusses lengthways of the shop on the traveler, and the other motion across the traveler by means of a trolley car. The amount of work which an ordinary riveter will turn out depends almost entirely upon the methods of handling the work at the machine. An ordinary riveter makes from sixty to one hundred strokes per minute, and its ability to drive a given number of rivets is limited only by the ability of the operator to get the work to be riveted, with a hot rivet inserted, between the dolly bars ready to be headed. The student will therefore understand from this that the number of rivets driven, or the cost of production in this part of the plant, is dependent entirely upon the appliances for handling the work. When the work exceeds two tons in weight it is generally advisable to move a riveter about it. The portable riveters are run by compressed air, the power being carried through a rubber hose, strengthened and protected by wire or marlin; in this way the riveter with its power can be moved about the work.

Fig. 48 shows an interior view of the Main Truss Shop, taken from a photograph, the camera standing at the finishing end of the building. The necessary appliances for moving work about the riveting machines, and the riveting machines about the work, are very effective. They are strong, rigid and

FIG. 48.

capable of quick action, as a large amount of time and unnecessary labor may here be consumed without profitable results. In a shop like this where all classes of structural work are manufactured, from the heaviest railroad bridges to the light iron roof trusses, it is necessary that the appliances for handling the work in this part of the plant be of peculiar adaptability; heavy and strong for the heaviest work, and light, quick and easily operated for the light work.

From the riveting machines the top chords and web posts of pin-connected work are delivered to the rotary planer, and thence to the drill presses; they then pass out of the shop into the yard to be painted and loaded.

The loading appliances are shown in the general plan on Fig. 46. They consist of two jib cranes connected by wire rope to the hoisting engine which is provided with four drums, so arranged that each drum can act separately and independently of the others. Each derrick is provided with two lines, one for raising and lowering the load, the other for raising and lowering the boom. By this arrangement one man operating the hoisting engine can work both derricks at the same time, and thus can raise or lower, or move the load out or in, as occasion may require.

This, briefly, is the course which work takes in passing through the truss shop at the works of The Berlin Iron Bridge Co. This shop, as before noted, is 80 feet in width and 400 feet in length. The general features of its construction are shown in the interior and exterior views, Figs. 48 and 49. The sides of the building are made of glass for a distance of 10 feet down from the eaves. Below this the sides are left entirely open in summer to allow free intercourse between the inside and outside of the building, and also for ventilating purposes. Ventilation in this part of the plant is of equal importance with the forge shop, for here the gases, smoke, dust and dirt from the riveting furnaces is as great as from the forges. The plan adopted is believed to be the best. The greater portion of the work in a plant of this kind is done during the summer months, and at that time the temporary sides may be left off entirely from the building. This admits of free circulation of air, and secures what might be called absolute perfection as regards ventilation. In the building referred to, during the hottest months of summer the air is cool and a light breeze is always perceptible. In this way the comfort of the workmen is insured, and the consequent production increased. Besides light from the glass sides, the interior of the building is lighted by a line of skylights 12 feet wide placed in the plane of the roof, extending the entire length of the shop. At night the whole building is lighted by 12 arc lamps and 250 incandescent lamps, all on the same circuit and driven by a Thompson-Houston dynamo. Arc lights, besides lighting the interior, are used to light the yard, while each machine has two or more incandescent lamps for the use of the operator. The building is heated by the Sturtevant system of hot air, and all furnaces, both for the boilers and rivet heating, are equipped with fuel-oil burners using crude petroleum. Ventilation is secured by a monitor at the ridge of the building, with swinging shutters on each side

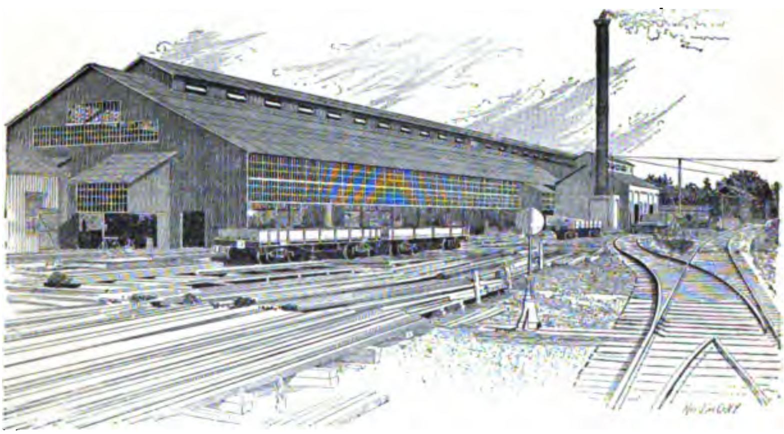

FIG. 49.

the full length of the building. The construction of the building is entirely of iron, no woodwork being used about it, so that the risk from fire is entirely eliminated. The cost of maintenance of the building consists only of painting and glazing, and is thus reduced to the absolute minimum.

The main source of power is a shaft extending the entire length of the building in the immediate center, placed on the upper side of the tie beam of the roof trusses. From this main shaft countershafts are driven which, in turn, furnish power for the individual machines. Travel crossways of the building is by means of trolley cars attached to the roof trusses. Motion lengthways of the shop is by means of overhead travelers, shown in Fig. 48, although most of the material is moved in this direction on the three narrow-gauge floor tracks. The student will notice in Fig. 48 a number of finished bars occupying the skidways. The drill press at the left is at work on an I beam, the one at the right boring a pin hole in a chord section. Directly over the latter will be seen an overhead track of 8 feet gauge, with a car. Upon this car is attached an ordinary chain hoist, which travels on the car crossways of the main track. In this way the operator has control of a floor space 120 feet long and 8 feet wide, and can take up a piece of iron at any point and deliver it wherever it may be required within control of his tracks. In the background are seen planing and riveting machines with numerous workmen busy at their several duties.

At the south end of the shop is the office of the superintendent and clerks, where are kept the daily accounts both of men and materials. Every pound of material which enters the works is carefully weighed on the cars before it is unloaded. When shipped out it is again carefully weighed and checked before being loaded. Two pairs of scales are provided for this work, one a track scale having a capacity of 120000 pounds (besides the weight of the car) and the other a 20-ton ordinary platform scale, which is placed on the center track at the south end of the building, the beam of the scale being in the superintendent's office, where the weighing clerk checks his weights as the material passes out.

## ART. 87. ROOF OF THE TRUSS SHOP.

The roof is supported by 26 trusses placed 16 feet apart between centers, each truss being 81 feet in span from center to center of supporting columns. The trusses are of the triangular type, 20 feet 2 inches rise to the ridge, where a ventilating truss is placed. Plate VII gives a general plan of the most important details of these trusses, together with the details of the overhead travelers. A transverse section of the boiler house with its roof trusses, $29\frac{1}{2}$ feet span, is also shown. All the joints are riveted, and the manner in which the splice and reinforce plates are arranged should be especially noted by the student. The lower chord of the main truss is a built-up I-beam section formed of a web plate 16 inches deep and $\frac{1}{4}$ inch thick, with four angles $2 \times 2 \times \frac{1}{4}$ inch at the north end of the building, and $2 \times 2 \times \frac{3}{8}$ inch at the south end of the building. This lower chord is made extra heavy to stand the cross strain from the travelers and trolleys, which subject it to excessive concentrated loads. It is supported at frequent intervals by the angle web bracing, in order to transmit the stresses to the rafter and side posts. The rafter is formed of a web plate $10 \times \frac{1}{4}$ inch, with two angles $2\frac{1}{2} \times 3 \times \frac{1}{4}$ inch. The arrangement of the purlins and the connection of the glass and corrugated-iron covering will be readily seen from the several drawings on Plate VII.

The roof is covered with The Berlin Iron Bridge Company's anti-condensation corrugated-iron covering. Corrugated iron is one of the best coverings for a manufacturing plant. It is absolutely fire-proof and, from the nature of its construction, when properly applied, will not warp, twist or work loose. The principal objection in our northern climates has been on account of the drip or condensation of moisture on the under side. In winter weather, as the result of the cold air on the outside of the building, the warm air in the inside becomes chilled and deposits a portion of its moisture (held in suspension) on the inner or under side of the corrugated iron, where it drips down on the work and material in the interior of the building. A large amount of time and inventive talent has been expended to obviate this trouble, but nothing has been found practically effective until the present arrangement adopted by The Berlin Iron Bridge Company, and used by them in the construction of roof coverings on their buildings.

The roof trusses, as above noted, are 16 feet apart, connected by angle-iron purlins. These purlins are placed about 27 inches apart, and upon them is spread one layer of galvanized-wire netting, stretched tightly and well laced together. Next are placed two layers of asbestos paper, well lapped, with broken joints. Above this are placed two or more layers of tarred paper, also well lapped, with joints well broken. Upon this surface the corrugated iron is laid in the usual manner. The asbestos paper and the tarred paper prevent the chilling action of the exterior air, and if any moisture does condense on the under side of the corrugated iron the tarred paper prevents its percolating through. The method here described has been in use on this building for four years, and no trouble whatever has been experienced from condensation or moisture on the under side.

The data assumed for computing the roof trusses were as follows: Dead load of the roof 10 pounds per square foot, snow load 10 pounds, and wind load 10 pounds, making a total of 30 pounds per square foot. This load under ordinary conditions would be somewhat light for a building located in the climate of East Berlin, but when it is considered that, besides this, the trusses in the north end of the building are calculated for a live load of three tons at any point of the lower chord, and the trusses at the south end of the building, in the vicinity of the riveters, planers and drill presses, are calculated for a load of five tons at any point in the lower chord, the dead, snow and wind load of 30 pounds per square foot may be considered sufficient.

The maximum stresses in the principal members being determined from these loads the sections were designed, using a working stress of 12 000 pounds per square inch in tension and 9 000 pounds per square inch in compression.

## ART. 88. WEIGHT AND COST OF TRUSS SHOP.

The materials used in the construction of this truss shop are steel and iron,—steel being used in the compression members, and iron in the tensile members. The weights of the different parts of one of the heavy trusses and the weight of one panel of the building are as follows:

| | Pounds. |
|---|---|
| Lower chord | 3 700 |
| Web members | 1 800 |
| Rafters | 2 450 |
| Ventilator truss | 425 |
| Posts supporting truss | 1 060 |
| Purlins (3800 on roof, 1300 on sides) | 5 100 |
| Bracing, average | 900 |
| Glass covering of roof, 256 square feet | 900 |
| Glass sides of building, 320 square feet | 900 |
| Iron covering of roof, 1480 square feet | 1 850 |
| Total weight for 16 feet in length | 19 085 |

The weight of the building per square foot of floor surface is hence nearly 15 pounds, while the weight of the trusses and roof is about 11 pounds per square foot of roof surface.

The cost of a building of this kind complete, without any of the internal appliances, as travelers, cranes and tracks, is approximately forty thousand dollars. This is about 8.4 cents per pound of material, or 125 cents per square foot of floor area.

# CHAPTER XI.

## SHOP PRACTICE.*

### ART. 89. PRELIMINARY. STANDARDS OF MEASURE.

WHEN the computers have figured the dimensions and weight of a structure and the drafting department has from these computations made the working drawings and from them, again, the bills of material and lists for shipment, the material is ordered from the mills and the drawings handed over to the shops to put into metallic form what has heretofore existed only on paper.

A through railroad bridge of the pin-connected type gives as good an example of the different classes of work to be done in the shop as any other structure that can ordinarily be found, and we may briefly sum them up as follows: Riveted work, which includes the compression members, floor and bracing; eyebars, constituting the pure tension members; forgings, embracing the clevises, bent yoke plates and loop eyerods; machine work on the pins and rollers; and foundry, in the cast filling rings on pins. Incidental to the riveted work is the making of templets; and belonging to the whole we have the inspection, shipping and painting, and finally the cost.

The fact of complete working drawings and bills of material reaching the shop means that everything is now ready for

* Written by SAMUEL TOBIAS WAGNER, M. Am. Soc. C. E., formerly Superintendent of Shops, Phœnix Iron Company.

all the dimensions to be transferred, full size, upon the material, and as the different parts of every metallic structure are made in different departments, conformity in the standards of measure is of vital importance.

Ordinarily there are two kinds of standard measures in use in most shops, one made of wood and the other of steel. A Brown and Sharpe two-foot scale is considered a standard for this length, and from a pair of these a standard measure should be made, one in wood and one in steel; and these standards should never be taken from the tool room of the shop. The wooden standard should be made of the straightest-grained, thoroughly-seasoned white pine obtainable, and kept well covered with a good coat of shellac varnish to exclude any moisture. This should be about twenty to twenty-five feet long, from ten to twelve inches deep, and three inches wide, mounted on legs, and graduated on the upper edge with the two steel scales at a temperature of about 70° F. Wood will shrink endways, and the standard should be verified from time to time and regraduated if necessary. The steel standard measures are best made ten feet long, of about three by one quarter inch metal, and laid out with the two steel scales when all are at the same temperature, in the same manner as the wooden standard. All poles issued to the shops should be made from these standards, and compared with them at regular intervals. It is customary to graduate all shop poles in inches, all fractions required on the work being laid off by a steel scale and dividers from the steel poles, and by an ordinary two-foot rule from the wooden ones. The standard measures therefore need only be graduated to inches. Shop men always use eighths, sixteenths, thirty-seconds, and sixty-fourths of an inch, and never decimals. Steel tapes are very convenient for checking dimensions, but should never be used to replace the steel poles where great accuracy is required.

## ART. 90. TEMPLETS.

A templet, in bridge or roof construction, is understood to be a frame of wood made of boards, which is an exact full-sized representation of the piece of metal required, and from which the location of all holes and cuts is made. Templets differ from patterns in that the former always apply to wrought iron and rolled steel, while the latter are used to mold up cast iron or cast steel. The templet shop is the first place the drawings of riveted work reach after being distributed among the shops. Templets are made to facilitate the transfer of the dimensions given by the drawing to the metal, and therefore the larger the number of pieces to be made from one templet the more economical is the cost of the templet, and when once made correctly there is very little danger of a mistake in punching. If there are only a few pieces of a kind to be laid out the practice differs ; some shops making a templet ; and others laying off one piece of metal in the shop and marking the others from this, or, where this cannot be readily done, in irregular shapes, laying out each individual piece separately. Shops equipped with multiple punches abbreviate their templet work materially, as very nearly all the holes are laid off by the machine. Where slight differences exist between two pieces, one templet, with the addition of certain notes, can often be made which will apply to both.

Templets are ordinarily made of well-seasoned white pine, surfaced on both sides, and of the commercial thickness, about seven eighths of an inch. While this thickness is much in excess of what is required, it is generally used on account of the ease of obtaining it and its comparative lower cost compared with thinner boards. Templet makers must be able to thoroughly and intelligently read the most intricate drawings, and they therefore command high wages. Compared with pattern makers, the templet maker has to make probably one hundred vital measurements to one of the pattern maker, and while the latter must have more ingenuity to know just how he shall best make and part his pattern so as to mold easily and cheaply, yet he quickly learns this, and it is probably more than offset by the fact that not a single dimension of the templet maker can be incorrectly made without serious results, while in the pattern only the principal dimensions need be exact. The making of templets is therefore an expensive operation. and all that can be done in the design to duplicate pieces, or to make the work symmetrical, decreases its cost.

## ART. 91. RIVETED WORK.

Everything is now ready for the shops except the material, and this will at present be assumed to have been rolled, inspected for surface defects and delivered to the shops. That class of work which must be connected with rivets, either in the shop or in the field, will first be taken up and followed through its various processes.

**1. Straightening and Curving.**—All material should be perfectly straight at the beginning, so as to insure first-class work in the end, and to accomplish this it must be examined and correctly straightened. Plates are more likely to be out of line than almost any other shape, and as they, on account of their great stiffness in the direction of their width, largely determine the outline of the member of which they form a part, it is of the greatest importance that they shall be straight. Several methods are in use which vary in excellence in the order here given.

Plate rolls are used with generally five or seven rolls arranged in two horizontal lines the one over the other, and so that the plate in passing between them is freed from bends, and by a special device it is also stretched on the short side and thus straightened when it is out of line in the direction of its width. These rolls do excellent work on plates of all sizes.

Trip hammers are used with carefully prepared hammer faces, so as not to cut the plate or seriously mark it. These hammers straighten the plate in the direction of its width by stretching the plate on its short side, and in the direction of its thickness by hammering out the bends. The use of sledges operated by hand is the most expensive and probably the most damaging to the material of any of the methods mentioned.

The action by hand is exactly similar to that under the trip hammer. Beams, channels and tee bars are best straightened under either a cam, hydraulic or screw press, and angles in the same way, or preferably through a set of grooved rolls. As a rule these shapes come from the mill in fairly good form and do not need much extra work. All curving is either done under a press, in cases where the radius is long; and when the radius is short the shapes are heated in a furnace and bent to a wrought- or cast-iron form, which is fastened on a large cast-iron plate such as is used in shipyards. When such curving is required, it is done before punching, so as to avoid the distortion of the holes, and to prevent their being stretched out of position.

When it is important that such curved material must closely retain its shape, the holes must be drilled after it is set to the exact curve, and not punched, as the latter will always distort its form to some extent. Small plates such as pin plates and battens in some shops are straightened, or bent when required, by heating in a furnace and while hot pressing them between dies, one of which is stationary and the other acted upon by an hydraulic piston.

**2. Cutting.**—The material is next cut to exact dimensions

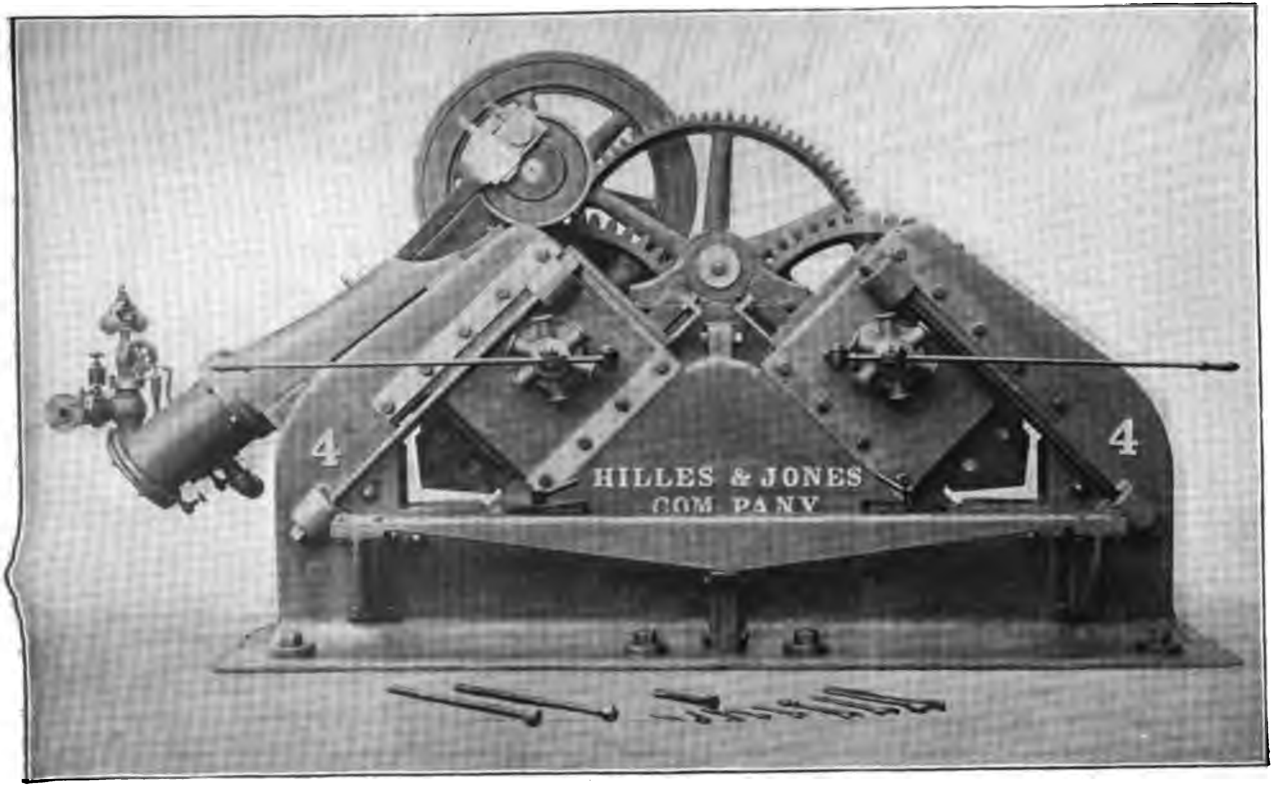


Fig. 50.—ANGLE SHEARS.

except where a machine finish is required, in which case the bars are cut from $\frac{1}{8}''$ to $\frac{1}{4}''$ longer on each end that requires machining. This is in order to insure a flush surface on the finished member. The long bars which have square ends are generally cut sufficiently close at the mills, but may require trimming. The shorter pieces almost always come from the mills in multiple lengths and are cut at the shop. All level cuts must be made at the shops to insure the best work. Three different machines are preferred for doing this cutting: plate shears for all plates; angle shears for all angles; and cold saws for beams, channels, tees, zees, and all irregular shapes. Each of these machines is capable of making the ordinary cuts on all sections usually rolled, but the cut must always represent the path of a single intersecting right plane. Fig. 50 represents an angle shear capable of making square or bevel cuts. The back of the angle must always rest against the stationary horizontal and vertical knives during the operation of cutting, and the horizontal leg kept level. The movable knife begins cutting in the fillet of the angle and then shears evenly on both legs. The bar may be moved to the right or left with the horizontal leg again remaining level, and then cut to whatever bevel may be desired, of course always within certain limits which depend upon the special construction of the machine. A bar placed in the knives on the left-hand side of the machine will be cut like the piece marked "left" in Fig. 51, and one cut on the right-hand side like that marked "right." Both of these are shown in horizontal projection.

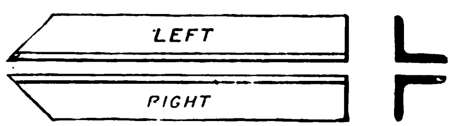


FIG. 51.

From the above is readily seen the impossibility of making a re-entrant cut. The compression of the shear knife always comes on that part of the bar which is in the rear of the machine, and is so great as to distort the shape of the angle when the length of the piece cut off is less than the width of the narrowest leg. It will be noticed by referring to Fig. 50 that the shear is driven by a special engine, a method which is economical and generally adopted.

The cold saw consists of a circular disk of steel about 44″ in diameter and $\frac{5}{16}$″ thick. It is mounted on a heavy cast-iron frame, and the journals in which the shaft carrying the saw runs are very long. In front of the saw is a carriage which moves upon V's, and upon which the shape to be cut is clamped, after which the carriage is fed up to the saw by hand through a hand wheel and rack and pinion. An engine making about two hundred revolutions per minute runs the saw and is so belted that a point on the circumference of the saw has a velocity of about 4 miles per minute. The saw cuts its way through the piece by melting the metal. The use of the cold saw is restricted to the cutting of shapes that cannot be sheared without expensive machinery and a great many changes of the cutting knives. Other cuts consisting of re-entrant angles can only be made by heating them in a smith fire and cutting them with a chisel, or by punching or drilling and then dressing up the edges cold with a hammer and chisel. Either method is expensive and to be avoided wherever possible Usually all irregular cuts on beams, channels, etc., are made from a templet.

**3. Punching or Drilling.**—For this purpose the templets are applied, clamped fast, and the holes transferred to the metal by means of a center punch and hammer. The holes in the templet are bored so that the center punch fits them snugly. The operation of making the holes in the metal after their positions have been located is done in two ways, either by punching or drilling. Punching consists in forcing a piece of hardened steel, which has the shape of the hole, through the metal in question by means of pressure exerted upon it in a machine, the metal punched out being forced through a die the top edge of which acts as a shear in combination with the punch.

Punching machines are either arranged to punch one or more holes at a time, and are known as single or multiple punches respectively. The single punch always works upon material which has previously been laid off by a templet or directly upon the metal, and is generally arranged so as to reach to the center of wide plates, and to punch all irregular shapes. A punch proper with its accompanying die is shown in Fig. 52. The tit on the bottom of the punch is for the purpose of centering the punch by entering the hole made in the metal by the center punch. Fig. 52 shows the punch and die in section through the center. The relative standard dimensions as required by most specifications are the diameter of the punch $d' = d + \frac{1}{16}$ inch, and the diameter of the cutting edge of the die $d'' = d + \frac{1}{8}$ inch, where $d =$ diameter of the rivet. The piece of metal cut out by a punch is called the "punching," and when the section of the punch is circular it is approximately a truncated circular cone, the small diameter of which equals the diameter of the punch, and the larger the diameter of the die. Punches can be made of almost any reasonable shape, but, on account of the great expense of making complicated forms, are not much used on structural work. As each piece of metal is laid out it is marked with the contract number and either its individual mark or the mark of the member to which it belongs.

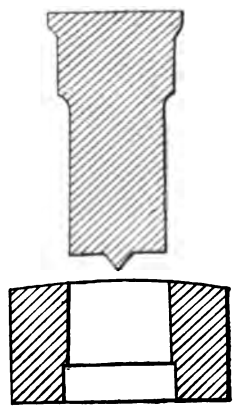

FIG. 52.

Fig. 53 shows the front view of a Sellers multiple punch without the carriage. Four punches with their dies are shown in the machine, and there is room for eleven more. These can be placed wherever required across the machine, which is capable of punching a plate 5′ 11″ wide. The gap shown in the center of the punch head is filled by a block when punching plates; it is now open for punching channels. Directly in front of the machine (not shown in Fig. 53) is a table which supports a spacing carriage. This carriage is peculiar. It has a spacing device which can with great readiness be set to any required distance in quarters of an inch from $\frac{1}{4}$″ to $13\frac{1}{2}$″ inclusive at a stroke. The carriage runs on rails and is propelled by means of a rack and wheel, and to it one end of the angles or plates to be punched are clamped. On the opposite side of the machine is a traveling carriage which holds the other end of the bars in line as they pass through the punch. It is thus clearly seen that all longitudinal dimensions are made by the spacing carriage in directing its motion and transverse dimensions by the setting of the punches. Multiple punches are generally used on the main members of the structure as stringers, floor beams and chords, where there are a great many rivets in line. They are capable of punching at one time four angles, or two channels, and several rows of rivets in a plate, and require practically no templets except for very complicated or difficult connections.

The work done by these punches is extremely good, mainly because they are more or less automatic. In order to simplify the work at these machines, and in fact as a general rule, it is advisable for the designer to confine himself to even numbers for rivet spacing. It is much easier for a workman to use integral numbers than mixed, especially where they repeat constantly. Multiples of $1\frac{1}{2}$″, 2″, or 3″ are the easiest to lay out with a duodecimal rule.

The practical limit of punching iron or steel is where the diameter of the punch equals the thickness of the metal. Exceptions to this occur, but it is neither advisable nor safe to count on them in general practice. Above this limit it be-

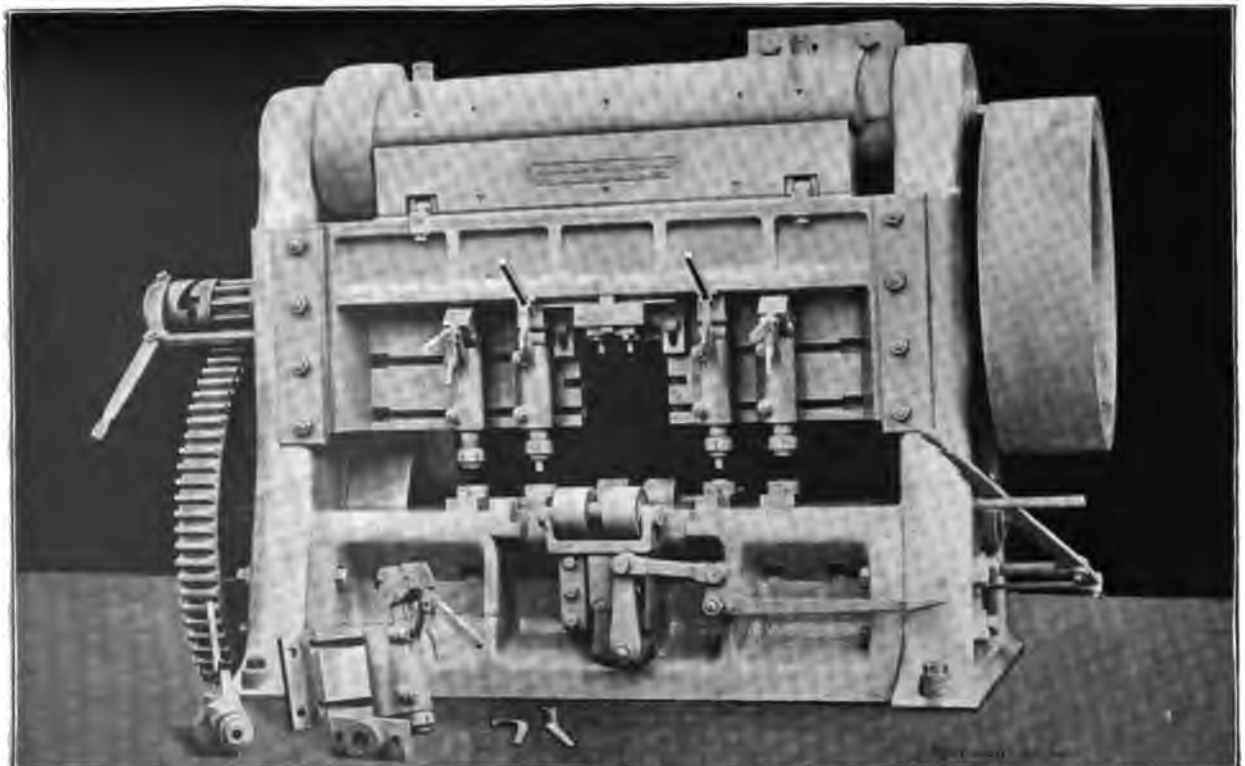

Fig. 53.—MULTIPLE PUNCH.

comes necessary to drill the work, which increases its cost very materially.

The work for a drill press must be laid out similarly to that for a single punch, and the tit on the end of the drill made to enter the center-punch mark. It will cost anywhere from three to ten times as much to drill an ordinary hole as to punch it, and as reaming a punched hole will not cost more than twice as much as punching, it will be cheaper as a rule on steel which must be punched and reamed or drilled to perform the former operations. Punching when carefully done is generally more reliable as far as accuracy of position is concerned than drilling. The drill may "run" or get out of place, while the punch if once properly centered will pass straight through. Of course the sides of a punched hole are not parallel, while those made with a drill are; this, however, is not material on riveted work. After punching or drilling all necessary countersinking must be done before the work passes to the assemblers. Multiple drills are used to a very trifling extent on bridge work in America. In England their use is much more general.

4. **Assembling and Reaming.**—All pieces which do not have riveting upon them now pass out of the shop as they are ready for the field; the remainder, however, are sent to the assemblers, where the component pieces of a member are fastened together with enough bolts to hold everything snugly in place, and the rivet holes which do not match sufficiently for the passage of a rivet are reamed. The machinery used for reaming varies in different shops, and numerous methods have been devised and are in use. The ends to be attained with such a machine are quickness in action while reaming, and ease in reaching all parts of the work. The arm of the machine which carries the reaming tool should have a reach of five or six feet on each side of the support so as to prevent frequent moving of the work; or better, be arranged so that the work remains stationary and the reamer moves. For intricate work

and on very large members that cannot be handled in the usual manner in the shop the Stow Flexible is very convenient. This reamer works with a flexible core in a leather shaft similar to that used by dentists, has its power transmitted to it by a rawhide rope, and is very rapidly and quickly handled by two men. For shops equipped with electric power, wires can be run to a small motor which drives the shaft direct.

In bolting up bridge chords and posts made of plates and angles, it is necessary to bolt up the plates and angles forming the sides of the member at one operation, and after they are riveted to have the cover plate and lattice bolted on as a final operation.

At this stage it becomes necessary to ream the holes in steel members where the specifications require the holes punched small and reamed in order to remove the metal injured in punching. There are two ways of doing this, some shops preferring to ream each single piece separately, while the others pass the reamer through after the work is assembled.

All holes which are left open for field riveting are now marked and a rigid inspection given the work to see that everything is in its proper place on the member.

5. **Riveting.**—The member now is ready for riveting, an operation which consists simply in plugging up the holes with rivets which are heated to a cherry red in suitable furnaces.

Several forms of supplying power to the cylinders of the riveting machines are in use, the principal being hydraulic power, from pressure pumps and accumulators; air, from air compressors; and steam, direct from the boiler. On work of large size it is neither profitable nor convenient to move the work to the machines, but is far better to bring the machine to the work, and for this purpose portable riveters are used. For the portable riveters compressed air is the most convenient power, as it is readily carried through a heavy rubber hose

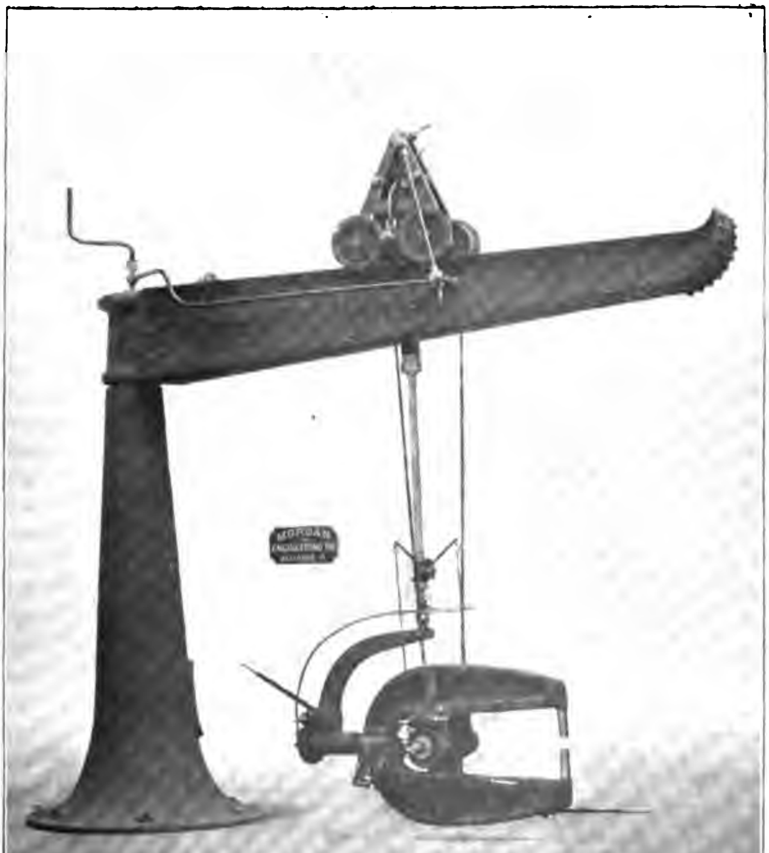


Fig. 54.—PORTABLE HYDRAULIC RIVETER.

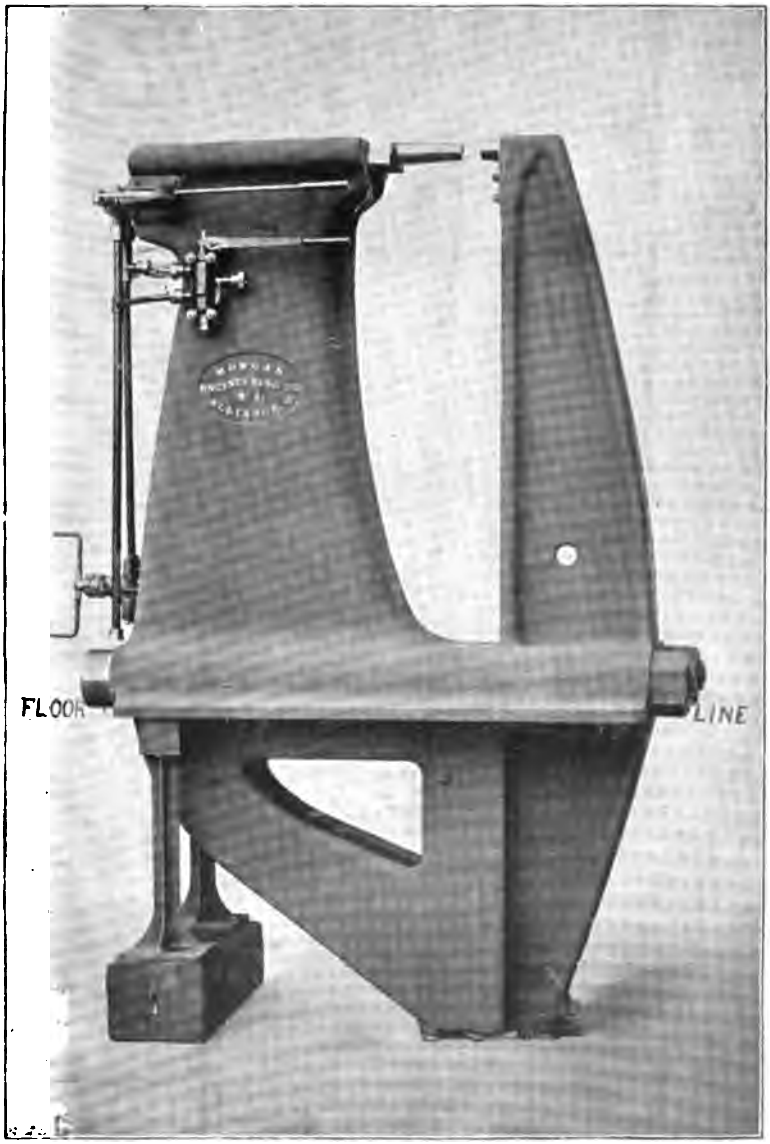


Fig. 55.—STATIONARY HYDRAULIC RIVETER.

which is generally wound with wire to make it durable. In order to take hydraulic power to a portable riveter it is necessary to use very heavy wrought-iron pipe with special stiff joints required by the pressure transmitted. From compressed air the pressure used is generally from 70 to 100 pounds per square inch, while the hydraulic pressure ranges from 900 to 3000 and upward. The diameters of the cylinders of the machines therefore vary greatly, and for very high pressures they are small and the machine correspondingly light.

Fig. 54 shows a portable hydraulic riveter mounted on a crane. The power is transmitted through the pipe shown on top of the crane, through the knuckle joints to the trolley and thence down to the machine. The riveter is shown in the proper position for riveting the cover plates on a girder or chord in their natural positions, but it can be turned to any angle whatever by means of the suitable apparatus shown. The hydraulic piston acts upon the lower jaw and forces it against the upper, thus pinching the rivet. The vertical height is readily adjusted by means of hydraulic pressure acting within the vertical tube. Any system of trolleys can be arranged so that the machine may run longitudinally over a large piece of work if necessary. The portable air riveter is very similar in application and differs only in details.

Fig. 55 shows a stationary hydraulic riveter. When in position the floor line is as shown. The work is necessarily carried overhead in the shop with trolleys upon a runway and suitable arrangements made to adjust the work vertically. The depth of the jaw through which the work passes is in some cases as much as ten feet, although five or six feet is the usual allowance and answers most purposes. The power is applied to the cylinder which is shown on top of the frame, and which operates the riveting cup shown on its end. The lever for operating is shown directly below the cylinder. The steam riveter is similar in its application, with the single difference of the substitution of a large steam cylinder for the smaller hydraulic one.

Hydraulic riveting is much to be preferred to any other on account of the almost constant volume which water maintains under different pressures, and steam is the most objectionable on account of its great expansibility after doing its work in the cylinder. A hydraulic riveter will do either its full work and maintain the pressure during the time it is applied, or it will absolutely refuse to work. With steam a sudden blow is struck with the full pressure, and the steam expanding makes the latter part of the stroke almost worthless, and whether the metal of the rivet has filled the irregularities of the hole remains uncertain. There are always some rivets in work which cannot be reached by power, and which must be driven by hand; and as a hand-driven rivet costs more than three times as much as one driven by power, these rivets should be avoided wherever possible in the design, outside of the fact that besides the extra cost they are very inferior in quality compared with rivets driven by power. Portable riveters as a rule are used in driving cover plates on girders, chords and similar work, while the stationary machine works on the web.

Rivet heads on structural work are as a rule nearly hemispherical, with a diameter of about one and one half times the diameter of the rivet. They are made by a machine which forms one head on the shank while the other is formed in the riveting machine; enough metal is allowed to extend beyond the metal of the work to form the second head in the riveting cup. The heads when formed should be concentric with the hole, and completely cover it; therefore if the punching is bad the riveting will always show it.

6. **Milling and Fitting.**—Members which must be brought to exact length are then taken to the rotary planers and the ends faced up, sufficient extra metal always being allowed in

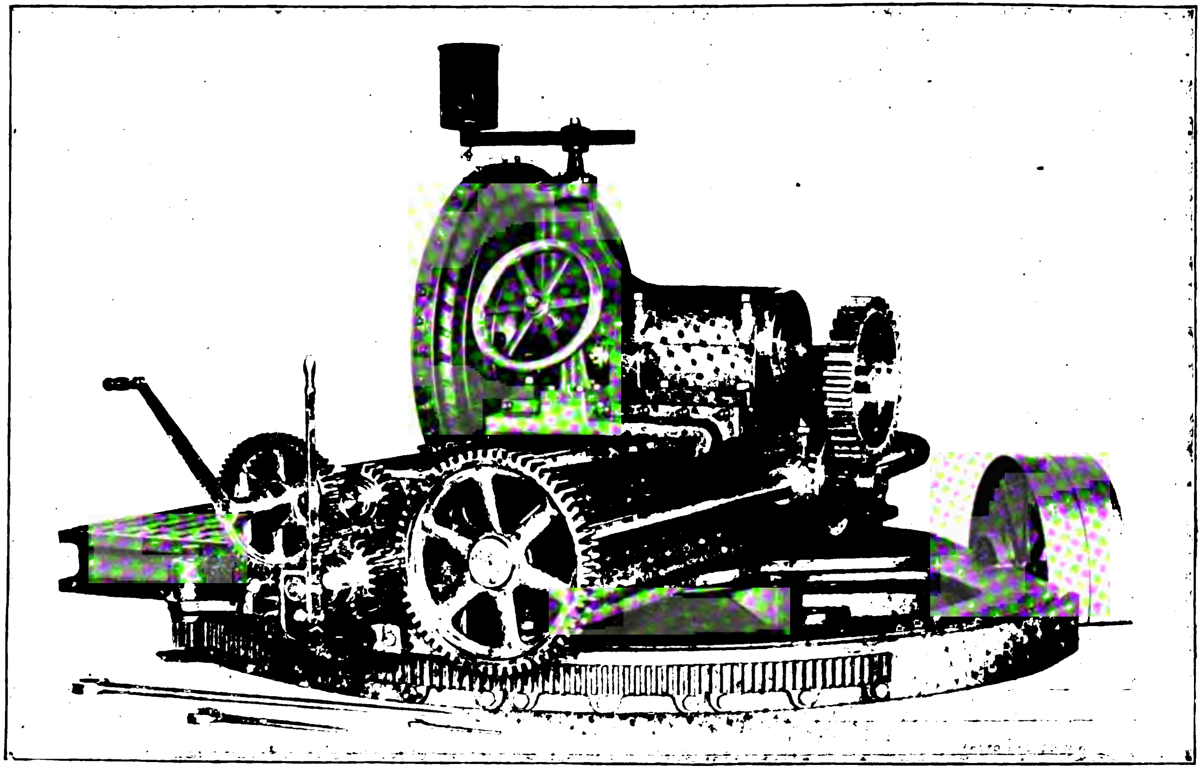

Fig. 56.—48-INCH ROTARY PLANER.

such cases to insure a true metallic end. In abutting members the greatest care must be taken in setting the work to make sure that the ends so faced are at right angles or at the proper angle to the axis of the member, and this, where there is no machined side or face to work from, requires considerable judgment on the part of the workman. At this point it becomes absolutely necessary to use metallic poles to make all measurements, as the metal in the member and the pole must expand and contract in the same ratio. Previously the conditions of the work have not required such accuracy, and the wooden pole has answered every purpose. It is always advisable in the design to have as few pieces as possible on the member projecting beyond the finished ends, as they invariably must be left off until after the member is faced, and then quite often it is impossible to rivet them by power, and the rivets must be hand-driven.

Fig. 56 shows a 48-inch rotary planer made by Bement, Miles & Co. of Philadelphia. The work is clamped upon the perforated bed plate shown to the left of the cut, which is also the front of the machine. It is set square to the face plate of the revolving head which carries a number of cutters fastened by the set-screws shown. This revolving head has a motion on the planed bed plate by means of feed gearing across the end of the work. The planed bed plate, head and all, move upon a central pivot, and are turned by means of a circular rack and pinion shown. This is for convenience in milling beveled work, in which case the center line of the work always remains at right angles to the normal position of the machine, and the member need not be skewed across the shop.

Shop practice differs in the methods of finishing the ends of stringers when they fit between floor beams, and of floor beams when they fit between posts, and where in either case the member must have an exact length back to back of connecting stiffeners. Some shops face the members before the end stiffeners are riveted on and after facing fit them to place, mark off and punch the holes in the stiffeners, and then rivet. Others order the metal from $\frac{1}{16}''$ to $\frac{1}{8}''$ thicker than called for in the stiffening angles, rivet them fast at the same time as the rest of the member, and calculate then to true up the ends in a rotary planer by cutting off all or a part of the surplus metal. Another method is to cut the chord angles and web plate short of the finished length, and then set the end stiffeners to exact length by measurement. In this case no facing whatever is done. Either of the first two methods is considered the best.

For pin-connected work the chords and end posts are now ready to go into the boring mill. When such members are punched the pin holes are cut to within about one half inch of the finished diameter of the pin. The boring mill consists of a long table with two movable head pieces. A longitudinal shaft transmits power to these head pieces wherever they may be. Each head piece has a spindle which may be either horizontal or vertical, and into this the boring bar with the cutters is fixed. Great care is taken in laying out the position of the pin holes, and the member is carefully lined up in the machine so that the center line of the hole is exactly at right angles to the axis of the member. When the holes are bored and before the member leaves the machine a gauge is tried in each hole to insure its being of the right size. This gauge represents the pin as to diameter, and the holes are usually from $\frac{1}{32}''$ larger, where the pins are over $7''$ diameter, and $\frac{1}{50}''$ in smaller sizes. There is now probably nothing to be done but to fit up the splices on chords and posts so that they will match in the field and at the same time guarantee an abutting joint; carefully chip and dress up all points of the work that require a neat finish. Everything is now ready for its final inspection where it will be left for the present.

## ART. 92. EYEBARS.

The tension members proper of a structure are usually round, square or flat bars. It is preferable to have round or square bars with loop eyes where the section is less than two and a half or three square inches, as they are cheaper and more likely to be perfect in workmanship; while above that point the flat eyebar with solid head is the best. It is again best for convenience of shops and mills to order the widths of eyebars in full inches and make up the exact section in thickness.

Iron eyebars are made either by upsetting alone or by upsetting and piling. Piling consists in placing about 30 per cent of the total metal in the eye upon the upset which represents the remainder, taking a welding heat in a furnace and hammering to shape in a die under a steam hammer. If the eye is composed entirely of upset material it is just as important that it receives a welding heat to unite the fibers before being hammered as if it were partly piled. This is practically accomplished by placing a small piece of metal on the eye when in the furnace, which metal must weld so thoroughly into the eye as not to be apparent after the hammering. After the eye is formed in the die a rough pin hole from $\frac{1}{2}''$ to $\frac{3}{4}''$ less in diameter than the finished pin is punched out under the hammer, when the forging is completed.

Steel bars must always be made by upsetting alone, as welds in structural steel are not allowable under any circumstances, while in iron the very metal itself is nothing more than a mass of welds. This upset, which is almost a perfectly formed eye, is made in a powerful machine which is operated by hydraulic pressure, is reheated and finished under the hammer in the same manner as the iron bar, or is then passed through a pair of flat rolls which smooth up the faces and reduce to its proper size the thickness of the eye, which is always in excess of the finished thickness as it comes from the upsetter. If any folds or cracks develop in the upsetting or hammering of the steel bar, or in fact anywhere in the manufacture, they must be chipped out until the metal is solid. They will never weld up and will always remain in the bar. The shape of the eye is usually circular, with the radius of the neck equal to the diameter of the eye. The excess of area in the head of an eyebar over the area of the bar when made of steel should never be less than 33 per cent, and generally 40 per cent is considered a good working limit. Steel bars with 20 per cent excess in the eye have broken the bar, but it is exceedingly dangerous practice. In iron 50 per cent is usually considered good practice.

The iron bars now go to a straightening press, where they are perfectly straightened and then sent to the boring mills. The steel bars go to the annealing furnace, where they are piled on edge with small intervals between, and after being inclosed in the furnace the whole bar is brought to a uniform cherry-red heat by gas fuel or wood. The fire is then allowed to die out, the bars cool slowly until cold enough to handle, when they are sent to the straightening press and thence to the boring mills. These mills are generally similar to those used in boring riveted work, except that in this case the boring bar is very nearly always vertical. In some shops special care is taken that after the machine is set for bars in one panel all of the bars of that panel are bored before the machine is reset. The bars now have their pin holes gauged and the machined surface covered with white lead and tallow to protect them from rust.

The pin hole must be as nearly as possible in the center of the eye, with its center on the center line of the bar, in order to be most efficient in the structure and to be really good workmanship.

Eyebars at the present time are made as large as $10 \times 2\frac{15}{16}$ inches; 12-inch bars will probably soon come into use. Bars of 4, 5, 6, 7 and 8 inches in width are the ordinary sizes used in bridge construction.

It is customary on all contracts of any magnitude, especially for steel bars, to break some full-sized eyebars in the testing machine in order to determine whether the method of manufacture and the annealing has been satisfactory. The strength developed in these tests has been shown to be slightly less than in the tests of small specimens.

The following is a list of the largest testing machines in the world capable of breaking full-sized eyebars of any of the larger sizes:

The Phœnixville machine at Phœnixville, Pa., the property of The Phœnix Iron Co., and the largest machine in the world. Its total capacity is 2 160 000 pounds, and it can break a bar 45 feet long with a stretch of 15 per cent. (See Engineering News, Dec. 28, 1893.)

The Athens machine at Athens, Pa., the property of the Union Bridge Co. Its total capacity is 1 244 000 pounds, and it can break a bar 40 feet long with a stretch of 12 per cent. (See Transactions American Society of Civil Engineers, Jan. 1887.)

The Watertown machine, at Watertown Arsenal, the property of the United States Government. Its capacity is 1 000 000 pounds, and it can break a bar 30 feet long.

The Edgemoor machine, at Edgemoor, Del., the property of The Edgemoor Bridge Co. Total capacity 700 000 pounds, and it can break a bar 35 feet long.

The Keystone machine, at Pittsburg, Pa., the property of The Keystone Bridge Co. Its total capacity is 600 000 pounds, and it can break a bar 30 feet long.

## ART. 93. SMITH WORK.

When square or round rods are used in a structure, and the connection is made by means of a nut on the end of the same, the ends must be sufficiently enlarged by upsetting so as to have an excess of area at the root of the thread over the body of the bar. This upsetting is done in a machine which upsets enough metal on the end of the bar to give the required section. The bars are then taken to a screw-cutting machine and threaded. If the ends of the rod connect to a pin, the rod is bent around the pin and welded on itself, forming what is technically known as a loop eye. In this loop the pull on the weld is indirect, which is its only redeeming feature, as a weld in the principal members of a structure which is subject to vibrations or shocks is not safe. In the case of counters where adjustment is needed, the loop-eye rods are cut in the body of the bar, the ends upset and threaded and connected by a turnbuckle or sleeve nut. The loop eyes are made in an ordinary smith fire and bent around a form by means of a lever; the end is then scarfed and carefully welded. They should always be bent around a diameter from $\frac{1}{16}''$ to $\frac{3}{32}''$ less than the diameter of the pin and bored out to insure a good bearing.

Most loop eyes when tested to destruction will break somewhere in the weld, and generally by tearing the weld open. They however, as a rule, develop nearly the full strength of the bar before so breaking. Care should therefore be taken to have these welds made by the most experienced men.

When a screw rod is required to take hold of a plate or angle a clevis is generally used. This is a forging made as follows, and is hammered from a solid piece of iron or steel. In Fig. 57 the first operation forms the nut, and partly shapes the neck; it also makes the hole in the nut. The second operation under another die forms the eyes of the jaws. The third consists in bending the eyes around the nut, and finishing the rough forging, which is then dressed up in a smith fire. The holes for the pin are then drilled under a drill press, and the nut tapped in a screw-cutting machine. Clevises are always shipped fast on the ends of the rods to which they belong. Bridge work may require forgings of a great variety, but most

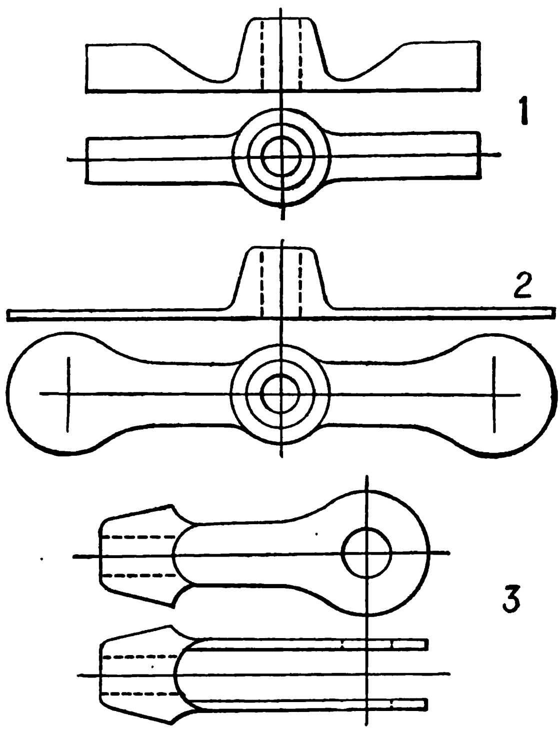


FIG. 57.

of them are resolved into ordinary smith or hammer work which it is not necessary to describe in further detail. Sleeve nuts and turnbuckles are specialties in the trade, and are seldom made by manufacturers of structures.

Bolts and rivets are usually made by most of the larger shops which do the heavier part of the work, and their object in so doing is to be able to control their own product. The machines in which they are made are upsetters operated by lever power and driven by belts and shafting. The heads are formed by upsetting the rod in properly-formed dies. Stocks of the sizes most frequently used are always kept on hand ready for use at a moment's call. The lengths of rivets generally run by eighths of an inch, and the diameters most in use are $\frac{5}{8}$, $\frac{3}{4}$ and $\frac{7}{8}$ inches. Shops ordinarily are not willing to make diameters in sixteenths of an inch on account of the necessity of making new tools.

Bolts in stock generally vary by quarters of an inch in length, and are of the same diameters as given above for rivets. They are usually made with hexagonal heads and nuts.

## ART. 94. MACHINE WORK.

Machine work proper on bridges and other structures (by which is meant lathe and planer work) consists principally in turning pins and rollers, and in planing wall plates, shoes and pedestals; while on swing spans there is the planing of the beveled track plates, the turning of the conical wheels, and the machinery for turning and locking the span. For pins and rollers the material comes to the shop from the mills from $\frac{1}{16}''$ to $\frac{3}{16}''$ larger in diameter than the finished diameter, and has to be placed in a lathe and turned to fit the gauge which now represents the size of the hole bored in the riveted member or eyebar. Cold-rolled iron and steel is being used to a considerable extent for pins and rollers up to $4''$ in diameter; it is beautifully polished by the action of the rolls, and so true to gauge that no turning is necessary. The ends of the pins are cut down, and the threads chased for the pin nuts, while the pin is still in the lathe. The planer is used for finishing up the bearings of the shoes which go on the roller frames and the wall plates on which the roller nests rest, and for planing a level in any plate where such level is required by the grade. The remaining work that must be done in the machine shop proper contains nothing of special interest, and outside of fitting up the machinery for swing bridges is mainly embodied in the above-mentioned operations.

## ART. 95. FOUNDRY WORK.

Ordinarily cast iron is not an important factor in bridge and roof construction, and hence it will not be necessary to go into details regarding foundry work. In swing span work there is generally a larger amount of work for the foundry than in any other style, as the top and bottom track, the conical wheels, the rack and pinion, together with the gearing that is necessary to turn and lock the span, all makes work for the molder.

The drawings for this class of work when being distributed among the shops all go to the pattern shop, where old patterns are either used, altered, or new ones made. These are then carefully marked and turned over to the foundry foreman. On structural castings the foundryman has to exercise special care, and mix his iron so as to produce a good, strong, tough casting. The track and wheels in swing-bridge work should be made of the best hard roll iron so as not to wear too rapidly. Designers should always avoid making castings where large masses of thick metal lie alongside of thin places. In such cases serious internal strains are developed in cooling, which if they do not break the casting at once, may do so at any moment, even after the casting has been cold for weeks. The foundryman takes all the precautions with such castings that he knows; after the casting has been poured he strips the sand carefully from the places where the thicker metal exists, and keeps that which is thin well covered up; but all his care is apt to count for nothing if the difference in thickness is too great. Sharp corners should be avoided, and all re-entrant angles rounded so as to enable the pattern to be readily drawn from the sand, and also to strengthen the casting. As a rule in castings, those which are specially smooth and in which the metal is run into the mold in a very thin condition are most deficient in strength, and the casting which has the most strength and toughness is liable to be rather imperfect on sharp corners and edges, owing to the fact that it is more viscid while hot.

All patterns are made from a shrinkage rule which is $\frac{1}{4}''$ long in $24''$; this is about the proportion in which cast iron shrinks in cooling from the liquid state. Any one measuring a pattern with an ordinary two-foot rule must bear this in mind.

## ART. 96. INSPECTION.

There are generally two kinds of inspectors around a bridge or roof shop, the first being the man appointed by the manufacturer, and the second the man sent by the purchaser or his engineer, whose representative he is, to see that the structure is built in accordance with the specifications and drawings.

The purchaser's inspector, when visiting a shop, should have in his possession a copy of the specifications, and obtain from the drafting department a full set of working drawings and lists of material. He should then proceed to find the names of the mills and their location from which the material has been ordered. He should place himself in communication with them, and be kept informed as to when they will roll the material for his structures. At this time also he should furnish them a list of the test pieces required by his specifications. When a rolling is made of any size on his schedule, the inspector should be on hand promptly and see that the bars are of the section required, select his test pieces if any have been ordered, and then examine every bar for surface defects. If the tests are satisfactory every bar which has been passed should be stamped with a distinguishing mark usually cut on one end of a small steel hammer, and the imprint on the metal surrounded by a ring of white lead so as to easily locate it.

To shopmen this stamp on the material is the signal that they can proceed with their regular shop manipulations without asking any questions. At many of the shipyards in England not a bar of metal will be received at the works without the inspector's stamp on it. This is an excellent plan as it places the inspection where it should be, that is, at the place where the material is made. This allows of the substitution of good for rejected bars with the minimum delay and expense to all concerned. Cases have been known where it has been necessary to reject a single bar out of a lot of material very urgently required three hundred miles away from where it was manufactured, and wait for a week or ten days until it was replaced.

After the material has reached the shop the inspector merely wants to watch the work as it proceeds through the various stages which have been heretofore mentioned, and see that the workmanship is good, and that the metal is not maltreated in any way. The inexperienced inspector may fall into a trap when he sees the drift pin, which is proscribed in his specifications, being used in assembling. It is just at this point that the drift pin is doing its legitimate work, where it is used in order to force the component pieces of the member into position before any bolts or rivets are in place. After the work is bolted together, reamed and some rivets driven the drift is doing dangerous work if used, as it is now enlarging the rivet hole at the expense of serious compression in some of the component pieces. There can be nothing but distortion, as the work is held by the rivets already driven.

As soon as the riveting is done the inspector should examine the rivet heads to see if they are full, well formed, concentric with the hole and tightly driven. This latter requirement is tested by striking the rivet a sharp blow on the side of the head with a small hammer. Practice soon tells which are loose and which are not. At this stage of the work it is well to raise any objections regarding straightness, as if this is not satisfactory it is the best time to rectify it. After the milling and boring are done and before the work is oiled or painted the inspector should satisfy himself that it is built according to the drawings and that the dimensions are as called for and then if satisfactory stamp it.

With eyebars he should keep a general lookout over the operation, but need make no actual inspection until the bars are finished; then, as he has previously examined the material in the body of the bar, he should measure the diameter of the eye and the thickness of the same and have pin gauges tried in the pin holes. The general finish of the eye should receive his attention and he should see that the neck has its proper section. Great care should be taken to observe any flaws which exist in the eye, especially in steel bars, as they are much more dangerous than in iron. Forgings and pins should be carefully looked over and measured and all finished surfaces covered with white lead and tallow to prevent rust. The inspector has no right to insist on any special method of manufacture at any stage of the work, unless such process is distinctly required in his specifications. All he is interested in is that the work complies with the specifications when it is completed, provided that the metal suffers no injury in the processes used.

The inspector acting for the manufacturer is not concerned especially until the material comes out of the shop, except in certain cases. Most manufacturers inspect themselves all material that they buy from outside their shops, no matter who the customer's inspector may be. Again, there are always regular rivet inspectors in each shop who see that no loose or faulty rivets pass the machines, and that the member is assembled properly before any rivets are driven. When the member comes out of the shop finished, the shop inspector is to be especially careful to see that all minor details and clearances are as shown on the drawings, and a good inspector will often find errors of clearance which exist on the drawings. He is always on the lookout for total and principal dimensions; and the location of all field holes, in fact the parts of the members where connections are made concern him most. He is making a critical examination to avoid any trouble to the erector in the field, as errors are much more cheaply corrected in the shop; and in the field delays may be very serious. If the work is erected by the purchaser his inspector should watch these points with more care than usual, so as to avoid this same possible delay in the field, but when the manufacturer erects the structure there is not as much need for the purchaser's inspector to be particular on clearances. The manufacturer is held responsible for any errors of his own making all the way through from the mill to the finished structure, no matter whether they have been passed by the purchaser's inspector or not.

An inspector should be on hand at all times, or within easy call, so that shopmen can consult with him about questionable points as they arise, and in this way avoid a great deal of friction which may occur if they proceed in the way that seems best to them without consultation with the inspector. When reading and acting upon his specifications he should always remember the note that THEODORE COOPER places at the head of his specifications for bridges:

"The most perfect system of rules to insure success should be interpreted upon the broad grounds of professional intelligence and common sense."

See an article by the author of this chapter in Transactions American Society of Civil Engineers, Dec. 1887.

## ART. 97. PAINTING AND SHIPPING.

When the finished member has been inspected and stamped, it is ready for painting and oiling. Specifications vary exceedingly in these requirements, but probably the following clauses taken from COOPER'S specifications cover good practice:

"All iron work before leaving the work shall be thoroughly cleaned from all loose scale and rust, and be given one coating of pure raw linseed oil, well worked into all joints and open spaces.

"In riveted work the surfaces coming in contact shall each be painted before being riveted together. Bottoms of bed plates, bearing plates and any points which are not accessible for painting after erection shall have two coats of paint: the paint shall be of a good quality of iron-ore paint subject to approval of the engineer.

"Pins, bored pin holes and friction rollers shall be coated with white lead and tallow before being shipped from the shop."

Some specifications call for red lead instead of oxide-of-iron paint, some for special trade paints and some for coal tar; some for a coat of raw linseed oil, and then a coat of oxide-of-iron paint; and, lastly, some for pure boiled linseed oil—all before leaving the shop.

When the work is painted or oiled the shipping department of the shop takes hold of it and places it on the cars ready to go to its destination. Power handling is advisable at all times, but more especially toward that part of the shop where the work is finished, as the member is constantly gaining in size and weight as it nears completion; therefore the shipping department necessarily handles the heaviest weights, and good facilities should always be at hand to place large members on the cars. When plate girders 120 feet long are shipped in one piece, and short members weighing 30 tons or more are to be handled, it requires considerable ingenuity to place them on the cars and securely fasten and brace them so that they will not be injured in transit. When the length of members exceeds that of one car, the bottom of the member must be raised high enough to clear the sides of the car by means of bolsters built up from its bed. On top of these bolsters it is customary to fasten an iron plate to reduce the friction of the member when the cars are passing around curves, and to prevent the metal of the member cutting into the wood of the bolster. Precautions must be taken to insure the piece from any longitudinal motion, as since the general introduction of air brakes on freight cars there is a decided tendency to move in this direction.

All bolts, field rivets and small pins are boxed and marked so that their contents can be readily known.

The limiting height of loading on cars above the rail varies on different roads, and the design must be kept within this height, or the member shipped in pieces and riveted in the field. In eastern Pennsylvania material which is 14 feet 6 inches above the top of the rail when loaded is about as high as the railroads will accept. This height depends upon the clear height of tunnels and bridges on the route, and therefore it is necessary to ascertain what can be transported over the desired route before finally settling on the design.

Before shipment the weight of every piece is carefully taken and entered on a record book.

## ART. 98. COST OF MANUFACTURE.

Every order passing through the shops receives an order number for easy identification, which is marked upon all drawings and bills of material, and in white lead upon all the material as fast as it is appropriated for the order. Whatever labor is done upon this material is reported to the accounting department from day to day, and when the contract is finished is all collected together. This labor account, together with the particular share of the cost of superintendence due the order, the steam, oil, tallow, interest on plant, cost of repairs, small merchandise, and the actual templet lumber consumed, is considered the cost of the work, bare of profit.

When the men in a shop work by the piece the cost of work of similar kinds should be practically the same on two different contracts. Under the piece-work plan, if it costs one cent to punch one hole it will cost two cents to punch two; and this proportion generally holds good whether the men work by the piece or not. Want of duplication or symmetry in the design very materially increases the cost of the templet-shop work, and the laying out on the metal, and on complicated work it takes the men longer to become familiar with the drawings; and by this is meant the number of different pieces to be built, and not the number of sheets of paper. The want of symmetry, however, does not affect the cost of assembling or riveting, unless the number of rivets in the two cases is different. A design which calls for such an arrangement of the component pieces that the majority of the work must be riveted by hand increases the cost by a very considerable amount. It is sure to run much higher than that on which machines can be used.

Ordinarily the cost of work increases in the same ratio as the difficulties in the design. If it takes long to work out a design on paper, it will take a correspondingly long time to work it through the shop, and the cost will increase in proportion.

It is impossible to assign any cost per pound for ordinary structural work on account of the great range and variety of the different classes of construction, the specifications, and the fluctuating values in the market prices of the material; but for bridge practice it will be economical in the shop as well as for other considerations to limit the length of plate-girder spans to 80 feet, lattice girders to 125 feet, and from that point up to use pin-connected spans.

The cost of any structure can be most accurately determined with the least loss of time by submitting the design to some manufacturer, or by a comparison with work of a similar style when such work can be found.

At the present time most millmen prefer rolling steel to iron, and the cost will not be essentially different in the shops unless the specifications require that the holes must be reamed, and all sheared edges planed.

An effort to keep the number of different shapes down to a minimum will do a great deal toward hastening the completion of the work, and will probably lessen the cost. For instance, use one width of eyebar throughout the bridge if possible, and use as few sizes of angles on chords and floor system as can be conveniently done; but changes in thickness do not matter. Keep flanges of plate girders horizontal instead of curving them as theory demands for uniform strength. Avoid blacksmith work wherever possible, as it is very expensive to make, and ordinarily if the same end can be accomplished in some other way it will be cheaper.

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